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BEM AND FEM IN ELASTO-PLASTICITY

STATIC AND DYNAMIC ANALYSIS OF THE SHORT WALLS WITH HOLES

BY

VICTOR FLORIN POTERAȘU and NICU MIHALACHE

1. Introduction

Static and dynamic behaviour of short walls of reinforced concrete or metall studies are necessary for the design of structures in seismic regime. By the dynamic analysis made by BEM we obtain the fundamental vibration mode and the stress values. We use a computer program for the dynamic study in the frequency domain with Laplace transform. Also, we make a dynamic analysis by FEM and a comparative study of the results with BEM. For example, we take a short wall with three storey and two holes for the doors. Around the holes appear the stress concentrations and by way of consequence the plastic hinges. In the plasticity range we will make by FEM a study of the cracks appearance and propagation simultaneous with the growth of the horizontal seismic forces. Also, we will make a study of the plastic range propagation and a dynamic calculus in the elasto-plastic range by NONSAP computer program. The bearing capacity of the short wall under the seismic load will be determined for some cuts of the structure and for the whole structure. In the theoretical part of the paper we introduce the algebraic-logical R functions to improve the results obtained by BEM.

2. Dynamic Analysis by Boundary Element Method with Laplace Transform

The equation of motion of the linear isotropic elastic body known as Navier's equations, are [2];

$$(1) \quad (\lambda + G)u_{j,ij} + Gu_{i,jj} + f_i = \ddot{u}_i,$$

or in form of propagation velocities [4];

$$(2) \quad (c_1^2 - c_2^2)u_{j,ij} + c_2^2 u_{i,jj} + F_i = \ddot{u}_i,$$

where c_1 and c_2 correspond to the propagation velocities of dilatational (P -wave) and distortional (S -wave) waves, respectively, given in terms of Lamé's constants G and λ as:

$$(3) \quad c_1^2 = \frac{\lambda + 2G}{\rho}, \quad c_2^2 = \frac{G}{\rho}, \quad F_i = \frac{f_i}{\rho}.$$

The constitutive equations are :

$$(4) \quad \sigma_{ij} = \rho [(c_1^2 - c_2^2) u_{r,r} \Delta_{ij} + c_2^2 (u_{i,j} + u_{j,i})]$$

and the boundary equilibrium equations are :

$$(5) \quad p_i = \sigma_{ij} n_j.$$

The Laplace transform with respect to time of a function $f(x, t)$ is defined as :

$$(6) \quad L(f) = \bar{f}(x, k) = \int_0^\infty f(x, t) e^{-kt} dt, \quad x \in [E^2, E^3],$$

where k is the Laplace transform parameter. From properties of Laplace transform, it is obvious that :

$$(7) \quad \bar{u}_i(x, k) = L[u_i(x, t)]; \quad \bar{t}_i(x, k) = L[t_i(x, t)]; \quad \bar{u}_{i,j}(x, k) = L[u_{i,j}(x, t)], \\ L[\dot{u}_i(x, t)] = k^2 \bar{u}_i(x, k) - k u_i(x, 0) - \dot{u}_i(x, 0).$$

Applying the Laplace transform to (2), (4), (5) and defining :

$$(8) \quad \bar{Y}_i(x, k) = \rho [F_i(x, k) + k u_i(x, 0) + \dot{u}_i(x, 0)],$$

yields :

$$(9) \quad (c_1^2 - c_2^2) \bar{u}_{j,i,j} + c_2^2 \bar{u}_{i,j,j} - k^2 \bar{u}_i = -\frac{1}{\rho} \bar{Y}_i,$$

$$(10) \quad \sigma_{ij} = \rho [(c_1^2 - 2c_2^2) u_{r,r} \Delta_{ij} + c_2^2 (\bar{u}_{i,j} + \bar{u}_{j,i})], \quad (11) \quad \bar{p}_i = \bar{\sigma}_{ij} n_j.$$

The second Maxwell-Betti theorem for elastic media, established between the Laplace transform of two systems, $(\bar{x}_i^*, \bar{t}_i^*, \bar{u}_i^*)$ and $(\bar{x}_i, \bar{t}_i, \bar{u}_i)$, gives :

$$(12) \quad \int_{\Gamma} \bar{t}_i^* \bar{u}_i d\Gamma + \int_{\Omega} \bar{x}_i^* \bar{u}_i d\Omega = \int_{\Gamma} \bar{t}_i \bar{u}_i^* d\Gamma + \int_{\Omega} \bar{x}_i \bar{u}_i^* d\Omega.$$

If this relation occurs between the following systems : a) system 1 — elastodynamic model ; b) system 2 — fundamental elastodynamic model with solutions defined by :

$$(13) \quad x_i^*(y) = \delta(x, y) e_i, \quad \bar{t}_i^*(y) = \bar{T}_{ji}(x, y) e_j, \quad \bar{u}_i^*(y) = \bar{U}_{ji}(x, y) e_j, \quad \bar{\sigma}_{ik}^*(y) = \bar{\sigma}_{jik}(x, y) e_j,$$

where $x, y \in [x] = [E^2, E^2]$, then equation (12) becomes :

$$(14) \quad \bar{u}_j(x) = - \int_{\Gamma} \bar{T}_{ji}(x, y) \bar{u}_i(y) d\Gamma(y) + \int_{\Gamma} \bar{U}_{ji}(x, y) \bar{t}_i(y) d\Gamma(y) + \\ + \int_{\Omega} \bar{U}_{ji}(x, y) \bar{x}_i(y) d\Omega(y), \quad x, y \in \Gamma \cup \Omega.$$

Similarly, the stress tensor for an internal point is

$$(15) \quad \bar{\sigma}_{ij}(x) = \int_{\Gamma} \bar{D}_{kij}(x, y) \bar{t}_k(y) d\Gamma(y) - \int_{\Gamma} \bar{S}_{kij}(x, y) \bar{u}_k(y) d\Gamma(y) + \int_{\Omega} \bar{D}_{kij}(x, y) \bar{X}_k(y) d\Omega(y),$$

where D and S are tensors which depend of fundamental solutions U and T [3].

By analogy to the static case, we can solve the integral equation :

$$(16) \quad \bar{c}_{ji} \bar{u}_i(x) + \int_{\Gamma} \bar{T}_{ji}(x, y) \bar{u}_i(y) d\Gamma(y) = \int_{\Gamma} \bar{U}_{ji}(x, y) \bar{t}_i(y) d\Gamma(y) + \int_{\Omega} \bar{U}_{ji}(x, y) \bar{X}_i(y) d\Omega(y),$$

with $x \in \Gamma$, $y \in \Gamma \cup \Omega$.

The boundary equation in two-dimensional elastodynamic case for the equation (16) without body forces is

$$(17) \quad \bar{c} \bar{u}(x) + \int_{\Gamma} \bar{T}(x, y) \bar{u}(y) d\Gamma(y) = \int_{\Gamma} \bar{U}(x, y) \bar{t}(y) d\Gamma(y),$$

which connects the transformed displacement and stress fields in the surface of system $\Omega \cup \Gamma$. The expressions for the transformed displacement and stress field for internal points can be obtained using Eqs. (14) and (15) :

$$(18) \quad \bar{u}(x) = - \int_{\Gamma} \bar{T}(x, y) \bar{u}(y) d\Gamma(y) + \int_{\Gamma} \bar{U}(x, y) \bar{t}(y) d\Gamma(y),$$

$$(19) \quad \bar{\sigma}(x) = - \int_{\Gamma} \bar{S}(x, y) \bar{u}(y) d\Gamma(y) + \int_{\Gamma} \bar{D}(x, y) \bar{t}(y) d\Gamma(y),$$

with $x \in \Omega$, $y \in \Gamma$.

3. Discretization of the Equations and Boundary Approximation

For the numerical implementation of the integral equations (17)... (19) it is necessary to discretize the boundary Γ of the domain Ω in elements. For two-dimensional domains, the boundary can be discretized in one-dimensional elements defined in the case of parabolic interpolation by three nodes. The cartesian coordinates x_i^n of each node are specified, therefore the cartesian co-ordinates of a point, nodal or otherwise, will be defined by the parametric equation :

$$(20) \quad x_i(\xi) = N_n(\xi) x_i^n, \quad (\xi \in (-1, 1), n = 1, 2, 3, i = 1, 2),$$

where

$$N_1(\xi) = \frac{1}{2} \xi(\xi-1), \quad N_2(\xi) = 1-\xi^2, \quad N_3(\xi) = \frac{1}{2} \xi(\xi+1).$$

For isoparametric elements, where the geometric interpolation functions as the variables coincide, the value of a variable Φ in a generic point x of the boundary will be expressed by [1]:

$$\Phi(\xi) = N_n(\xi)\Phi_n, \text{ with } \Phi = u, t,$$

where the shape functions $N_n(\xi)$ are defined in (20); Φ_n are the discrete values taken by the variable Φ at the nodes ($n=1, 2, 3$) of the considered element. The substitution of the boundary, Γ , of the system by another discretized formed by M boundary elements with n , ($n=3$), nodes in each of them, permit that equation (17) be transformed itself into the discretized one:

$$(21) \quad \bar{c}\bar{u}(x) + \sum_{m=1}^M \left(\int_{\Gamma_m} \bar{T} N_n d\Gamma_m \right) \bar{u}_m = \sum_{m=1}^M \left(\int_{\Gamma_m} \bar{U} N_n d\Gamma_m \right) \bar{t}_m,$$

where

$$(22) \quad \bar{u}(x) = \{u \ v\}^T, \quad u_m^T = \{\bar{u}_1 \ \bar{v}_1 \ \bar{u}_2 \ \bar{v}_2 \ \bar{u}_3 \ \bar{v}_3\}_m, \quad \bar{t}_m^T = \{\bar{x}_1 \ \bar{y}_1 \ \bar{x}_2 \ \bar{y}_2 \ \bar{x}_3 \ \bar{y}_3\}_m,$$

or in compact form

$$(23) \quad \bar{c}\bar{u}(x) + \sum_{m=1}^M \bar{H}_m \bar{u}_m = \sum_{m=1}^M \bar{G}_m \bar{t}_m.$$

Similarly, Eqs. (18), (19) are discretized as:

$$(24) \quad \bar{u}(x) = - \sum_{m=1}^M \bar{H}_m \bar{u}_m + \sum_{m=1}^M \bar{G}_m \bar{t}_m, \quad \bar{\sigma}(x) = - \sum_{m=1}^M \bar{S}_m \bar{u}_m + \sum_{m=1}^M \bar{D}_m \bar{t}_m.$$

4. Fundamental Concepts of the R-Functions Method

For a domain Ω let us consider a boundary value problem given in the local formulation [8], [9]:

$$(25) \quad Au = f \text{ in } \Omega \subset \mathbb{R}^n, \quad u \in X(\Omega), \quad l_i u_i = \Phi_i \text{ on } \Gamma_i, \quad (i=1, 2, \dots, m); \quad \sum_{i=1}^m \Gamma_i = \Gamma,$$

or in the equivalent global representation

$$(26) \quad \int_{\Omega} F(u) d\Omega + \int_{\Gamma} G(u) d\Gamma = \min_u.$$

Here A, F, l_i, G are operators defined over a space of functions defined in the domain Ω and on Γ respectively. The R -functions method is realized in two steps. In the first one the analytical formula $B(\Phi; w, \Phi)$ for the general structure of solution (GSS) is constructed according to the given boundary

conditions and the shape of the domain. The second step is of numerical nature and it begins with discretization of the yet undetermined „free” function :

$$(27) \quad \Phi = \sum_{k=1}^M \Phi_k N_k(x), \quad (k=1, \dots, M).$$

Consider a brittle cracked of short walls of reinforced concrete. Let us assume that the stress and displacement fields in it may be obtained from a solution of the well known 2-D elasticity problem

$$(28) \quad \nabla^4 F = 0 \text{ in } \Omega, \quad F = M(s) \text{ and } F_{,n} = P(s) \text{ on } \Gamma,$$

in terms of the Airy's stress function F . One of the possible ways to get the required GSS is to find it as a sum of three components $B = B_1 + B_2 + B_3$ that satisfy the following conditions :

$$(29) \quad B_1 = M, \quad B_{1,n} = 0, \quad B_2 = 0, \quad B_{2,n} = P, \quad B_3 = 0, \quad B_{3,n} = 0.$$

In order to determine the function B_1 we may use

$$(30) \quad l_i B(\Phi) = (\Phi)_i \text{ on } \Gamma_i, \quad (i=1, 2, \dots, m; \forall \Phi \subset m).$$

The function Φ , defined in Ω and strictly satisfying all boundary conditions Φ_i prescribed on Γ_i , ($i=1, \dots, l$), may be calculated with formula;

$$(31) \quad \Phi = \sum_{i=0}^l \frac{\Phi_i \prod_{\substack{j=1 \\ j \neq i}}^l w_j}{w_i + \prod_{\substack{j=1 \\ j \neq i}}^l w_j}.$$

Functions w_i are defined as mapping $\mathbb{R}^n \rightarrow \mathbb{R}$ that satisfies the equation $\Gamma_j^0 = \{x \in \mathbb{R}^n : w_j^0(x) = 0\}$, ($i=1, \dots, k$), where $\Gamma_i = \Gamma_i \cap \Gamma$ is the boundary common to Γ_i and Ω domain. Since $w_i \equiv 0$ is equivalent to $-w_i^2 \geq 0$, in order to find w_i^0 we might use the relation

$$(32) \quad w_i^0 = (-w_i^2) \wedge m_\Psi \Leftrightarrow w_i = w_i^2 \vee m_{\bar{\Psi}}.$$

Here Ψ is any mapping $\Psi: \mathbb{R}^n \rightarrow \mathbb{R}$ that satisfies the condition $\Omega_\Psi = \{x \in \mathbb{R}^n : \Psi(x) > 0\}$, where Ω_Ψ is defined by the relation $\Gamma_i^0 = \Omega_\Psi \cap \Gamma_i$. Thus, we will obtain a function Φ that meets the condition $\Phi|_{\Gamma} = M$, but $\Phi_{,n}$ may still be different from zero. It would be useful then to introduce the so-called normalized w_P function. A function w is normalized to the k order if besides $w_{N/\Gamma} = 0$ it satisfies the following conditions :

$$(33) \quad w_{N,n}|_{\Gamma} = 1, \quad \frac{\partial^i w_N}{\partial n^i} = 0, \quad (i=1, \dots, k).$$

We may now write $B_1 = \Phi - w_N \Phi^*$, in which Φ^* is any function defined in the domain such that $\Phi^*|_{\Gamma} = \Phi_{,n}$; we have then

$$(34) \quad B_{1,n} = \Phi_{,n} - (w_N \Phi^*)_{,n} = \Phi_{,n} - \Phi^*|_{\Gamma} = 0.$$

Introducing in (33) the operator

$$(35) \quad D_N = w_{N,xx} + w_{N,yy}$$

we may write $\Phi^* = D_N \Phi$ and $B_1 = \Phi - w_N D_N \Phi$. The second component of the GSS may be easily written as $B_2 = w_N P$, where P is any function defined over Ω such that on the boundary Γ it is equal to $P(s)$. The third component B_3 satisfying homogeneous conditions is $B_3 = w_N^2 \Phi$. Finally we may write the GSS in the form

$$(36) \quad B = B(\Phi; w_N, \Phi) = \Phi + w_N(P - D_N \Phi) + w_N^2 \Phi;$$

there is the last form involves a free function Φ while the others satisfy the nonhomogeneous boundary conditions. The R -function operator can be written for the usual domain and introducing it in the integral equation of BEM leads to the increasing of the solution accuracy.

4. Elasto-Plastic Dynamic Response of the Short Walls

4.1. Simple-Notched Beam

This application consists in calculating the stresses in the tip of notch for impact loading versus Laplace parameter, k . The model and dimensions are shown in Fig. 1 a. The model has been discretized in 50 knots and 21 parabolic elements. The material properties are: $\gamma =$

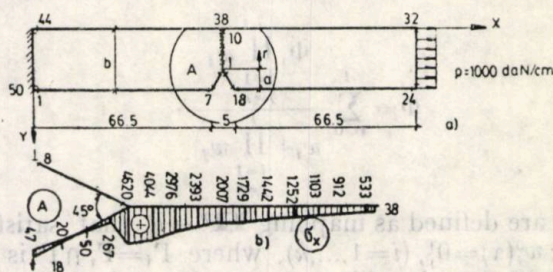


Fig. 1. — Dimensions and discretization studied.

$= 78 \text{ kN/m}^3$, $\nu = 0.3$, $E = 21 \times 10^{10} \text{ N/m}^2$. The stresses for internal points 1...10 to a distance a away from the crack tip are represented in Fig. 1 b. In the Fig. 1 b it is shown how the stress varies parabolically with the distance to the crack tip for $r/a \leq 0.28$ and linearly for a distance $0.28 < r/a \leq 1.72$. The stresses diagram from Fig. 1 b is obtained for a static analysis ($k=0$). The stresses and displacements variations obtained by a dynamic analysis with BEM and Laplace transform is shown in Fig. 2. The displacements variation u determined by means

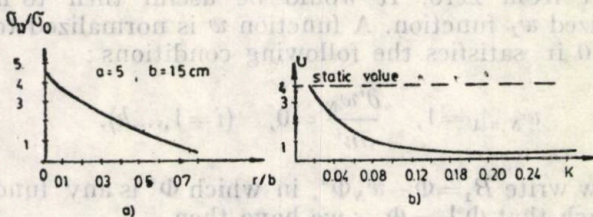


Fig. 2. — Stresses and displacements with BEM and Laplace transform.

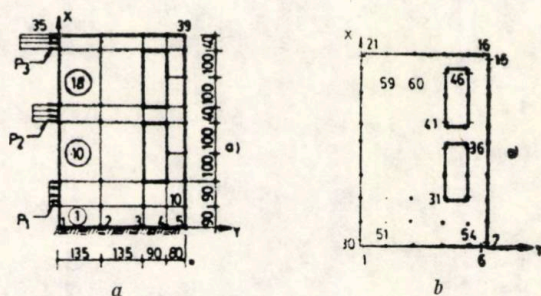


Fig. 3. — Discretization by FEM (a) and BEM (b).

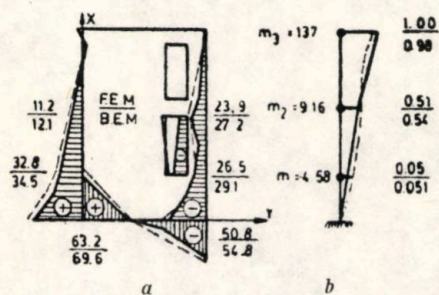


Fig. 4.—The state stress (a) by FEM— and BEM ----; the fundamental vibration mode (b) by FEM — and BEM-----.

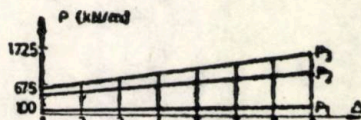


Fig. 5. — History loading.

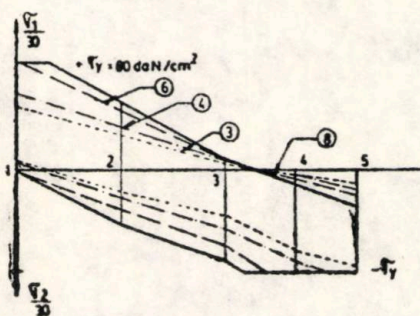


Fig. 6. — The state stresses in the embedded part.

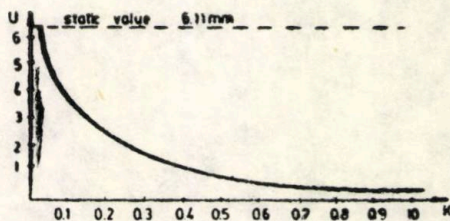


Fig. 7. — The displacement variation by BEM for the point 22.

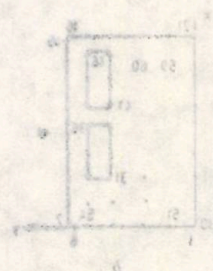


Fig. 1 - Discretization of FEM (a) and HEM (b).

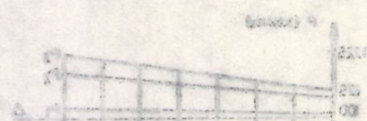


Fig. 2 - History loading.

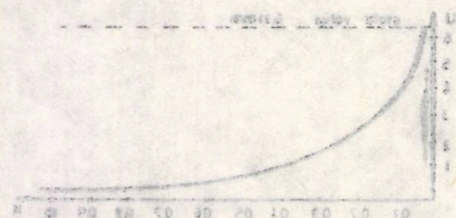


Fig. 3 - The displacement variation by HEM for the point 22.

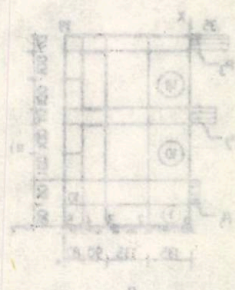


Fig. 4 - The state stress (a) by FEM and HEM; the fundamental vibration mode (b) by FEM and HEM.

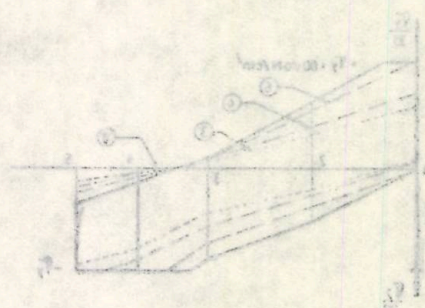


Fig. 5 - The state stresses in the embed bed part.

of Laplace transform is given for the points 24 ... 30 in Fig. 1 a. The maximum displacement value obtained by static analysis with BEM is $u=0.672$ mm and the exact solution is $u_e=0.657$ mm. It results an error of 2%. For values $k > 0.24$ the displacements are nearly zero. The BEM is more advantageous than FEM because it obtains the stress concentrations around the crack with a good accuracy and smaller computer time. Also, it results the appearance and the propagation of the plastic domain at the crack tip.

4.2. Static and Dynamic Behaviour of Short Walls

In this example we make a static and dynamic analysis by BEM and FEM.

The reinforced concrete of the seismic wall has the following elastic characteristics: $E=28 \times 10^9$ N/m², $\nu=0.16$, $\gamma=24$ kN/m³. The discretization by FEM and BEM is shown in Fig. 3. The thick of the seismic wall is $g=12$ cm. By FEM the concrete wall was been discretized in 24 quadrilateral elements and by BEM the reinforced concrete wall was been discretized in 50 knots and 31 linear elements. The stresses and displacements in both methods are obtained in the same points. In static regime and elastic domain the stresses was been determined for the following values of the forces: $p_1=100$ kN/m; $p_2=450$ kN/m; $p_3=675$ kN/m. The stresses values obtained by FEM and BEM on the boundary of the wall are presented in Fig. 4. For the normal stresses σ_x determined on the boundary of the wall it results the nearby values. The greater values obtained by BEM are situated in the corner of the holes. The static analysis of the wall by FEM is extended for the forces values until the appearance and the propagation of plastic hinges. The loads variation for the elastoplastic analysis, the von Mises law and the NONSAP computer program, is shown in Fig. 5. From the static analysis of the principal stresses σ_1 and σ_2 in the embedded zone it results that in the 3rd loading step the normal stress σ_2 reach the tearing limit of the concrete, σ_{cr} .

In the 4th loading step, in the compression part of the concrete appears the cracks. The cracks propagation leads to a plastic hinge in this zone (Fig. 6). At the beginning, the plastic hinges appear in the 15th element, then in the 9th and 12th elements which constitutes the column of the first hole. Then will appear three plastic hinges which transform the short seismic wall in a mechanism. Also, we make a dynamic analysis by FEM and BEM of the reinforced concrete wall. In the elastic analysis for the mass concentration values shown in Fig. 4 b was been obtained the fundamental vibration mode. The displacements values, obtained by both methods, are nearby. If we consider the impact force the displacement variation with respect to the k parameter is shown in Fig. 7.

5. Conclusions

BEM and FEM allow a comparative numerical analysis for plane structures in static and dynamic regime. The methods are advantageous reciprocally; BEM uses a more simple discretization and obtain with accuracy the stress concentration in the cracks area. The dynamic analysis by BEM with Laplace transform allows the impact problems study. The elastoplastic analysis made by FEM allows the appearance and propagation of crack study and the plastic hinges formation.

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METODA ELEMENTELOR DE FRONTIERĂ ȘI METODA ELEMENTELOR FINITE IN ELASTO-PLASTICITATE

Analiza statică și dinamică a diafragmelor scurte cu goluri

(Rezumat)

Se studiază comportarea diafragmelor din beton armat la acțiuni statice și dinamice în domeniul elasto-plastic cu metoda elementelor de frontieră și metoda elementelor finite. După ce se prezintă stadiul teoretic al problemei elastodinamice cu metoda elementelor de frontieră împreună cu un exemplu ilustrativ al unei bare din metal avînd o zonă de concentrări de eforturi se studiază propagarea zonelor plastice într-un calcul dinamic efectuat cu programul NONSAP. Pentru îmbunătățirea rezultatelor metodei elementelor de frontieră se introduc funcțiile algebrice-logice.

AN APPROACH AND AN ALGORITHM IN THE ANALYSIS OF MULTISTORY STRUCTURES AT SEISMIC TRANSIENT OSCILLATIONS

BY

DAN OLARU and LUCIAN STRAT

1. Introduction

According to the regular practice, the assumption of considering the simultaneous response of building floors when subjected to earthquakes has been largely generalized. This assumption, supported in most cases by the predominant fundamental vibration mode, of classical type, leads to the well-known form in the matrix equation:

$$(1) \quad [M]\{\ddot{x}(t)\} + [C]\{\dot{x}(t)\} + [K]\{x(t)\} = [M]\{1\}\ddot{u}(t),$$

where: $[M]$ is the inertia diagonal matrix, $[C]$ — the damping diagonal matrix, $[K]$ — the lateral stiffness matrix, $\{\ddot{x}(t)\}$ — the absolute acceleration vector, $\{\dot{x}(t)\}$ — the relative velocity vector, $\{x(t)\}$ — the relative displacement vector, $\ddot{u}(t)$ the motion acceleration.

In this way in expression (1) is neglected the time lag in which the seismic excitation propagates from the foundation level to the top floor of the building. This may result to a general distribution of the stress state for the structural members different from the actual observed one. This distribution, due to the interfering transient dynamic excitation, is specific to the phase of inducing seismic energy in all structural components.

2. The Background of the Procedure

Let be a multistory building with roughly a constant mass distribution over the height (Fig. 1 a). When subjected to lateral actions and neglecting the time lag, the fundamental mode is given in Fig. 1 b. Closer examination of long cantilever systems however leads to a different shape of the deflected shape (Fig. 1 c).

An analytical expression closer to the actual phenomenon requires the consideration of any time delay variable in both members of Eq. (1). So has proceeded R. M a r i n o v [1], in the case of an oscillating system of cantilever type with an uniformly distributed mass. In this way, for the multistory structures, with discrete mass distribution at floor levels, the matrix equation of motion considering the

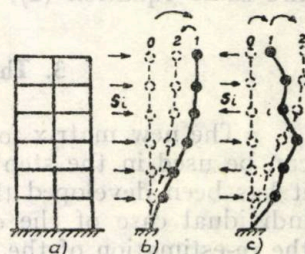


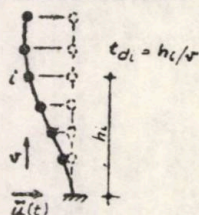
Fig. 1. — Modelling of vibrations at a multistory building : a) Plane structure ; b) Synchronous oscillation ; c) Nonsynchronous oscillation.

„nonsynchronism“ of the excitation — response function, can be written under the form :

$$(2) \quad [M] \{\ddot{x}(t)\} + [C] \{\dot{x}(t)\} + [K] \{x(t)\} = [M] \{\ddot{u}(t-t_a)\}.$$

The retardet arguments t_a represent the lag times of the response of different floors compared with the excitation, due to the propagation phenomenon, as illustrated in Fig. 2.

A special importance for the model has the determination of the parameters t_a . If the laws of the Theory of Elasticity are accepted on sub-ranges, in the case of the reinforced concrete, we have :



(3)

$$v = \sqrt{\frac{G}{\rho}}$$

and therefore

(4)

$$t_a = \frac{l}{\sqrt{\rho/G}}$$

Fig. 2. — Response delay in the transient phase.

where : v is the propagation velocity, ρ — the specific mas of the reinforced concrete, G — the transversal modulus of elasticity of the building material, l — the distance from the considered level to the foundation base.

The choice of relationship (3) is justified since the transversal wave propagation is taken into account. As regards modulus G , however, the values should be permanently reduced due to the increase of structural degradation. In these conditions, the response excitation lags will increase, consequently increasing the efforts. This phenomenon, not completely explained by means of the modal analysis theory, will be partly compensated by the increase of the structural damping, which will alleviate the overall response.

It will be noted that after a short time after stoping the seismic excitation, the building will oscillate freely; this is acurately described by the same equation (2), where the right member is zero.

3. The Description of the Algorithm

The new matrix formulation of the equation of dynamic equilibrium can be used in the step by step integration procedure. According to this, it has been developed the general algorithm presented in Fig. 3. For the individual case of the elastic oscillations, in the algorithm is eliminated the re-estimation of the rigidity, called also stiffness matrix and, implicitly, the comparison with the initial one („inelastic changes“ block).

The background of the algorithm, as already mentioned, is the modal analysis, which contains in the first stage the determination of the spectral matrices and eigenvectors.

As a result of the proposed algorithm, there are to be determined following matrices: the modal transposed $[\Phi]^T$, the generalized mass $[M^*]$, the generalized stiffness $[K^*]$:

$$(5) \quad [M^*] = [\Phi]^T [M] [\Phi],$$

$$(6) \quad [K] = [\Phi]^T [K] [\Phi].$$

At the same time there are to be determined:

a) The input accelerations for the coupled oscillations, considering the response delay

$$(7) \quad \{\ddot{u}_1(t)\} = \{f(\ddot{u}(t); t_d)\}.$$

b) The „external“ forces

$$(8) \quad [F(t)] = [M] \{\ddot{u}_1(t)\}.$$

c) The „generalized external“ forces

$$(9) \quad [F^*(t)] = [\Phi]^T [F(t)].$$

d) The input accelerations for the non-coupled oscillations

$$(10) \quad \{\ddot{u}_2(t)\} = [F^*(t)] [M^*]^{-1} \{1\}.$$

Following equation of motion corresponds in every r vibration mode:

$$(11) \quad \ddot{y}_r(t) + 2\nu_r \dot{y}_r(t) + \omega_r^2 y_r(t) = \ddot{u}_{2r}(t).$$

The step by step integration of these differential equations can be carried out in good conditions with the „exact method“, developed by Nigam and Jennings [2], resulting the response values z_r for every vibration mode separately, namely: displacement, velocity, acceleration.

For determining the actual response (where the oscillations are coupled), one must apply the superposition theorem. In this way, for the time t , the response z_i of the floor i can be approximated by a relationship of the form:

$$(12) \quad z_i(t) = \sum_{r=1}^n \Phi_{i,r} z_r(t).$$

4. Discussion of the Procedure

In the case when the multistory structure is characterised by a regular mass and stiffness distribution over the height, it may be accepted the

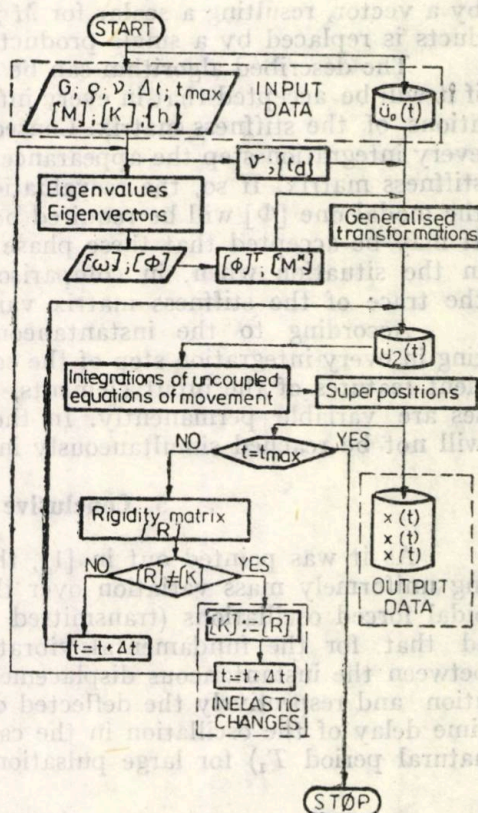


Fig. 3.—Flowchart for integration of the equations of motion for nonsynchronous oscillations.

consideration of the fundamental mode alone, thus resulting significant simplifications in relations (5) ... (10).

For instance, the square matrix of the eigenvectors $[\Phi]$ is replaced by a vector, resulting a scalar for M ; in relation (12), the sum of the products is replaced by a single product.

The described algorithm can be extended also for the nonlinear range if it will be accepted that in every integration step do not interfere modifications of the stiffness matrix. Consequently it is necessary to check after every integration step the appearance of incremental changes in the lateral stiffness matrix. If so, the reevaluation of the spectral matrix $[\omega]$ and of the modal one $[\Phi]$ will be required before going to a new integration step. It may be accepted that these phase come-backs are to be achieved only in the situation when, in comparison with the previous reference step, the trace of the stiffness matrix varies by 10%, 20%, a.s.o.

According to the instantaneous interstory displacements — resulting in every integration step of the equation of motion — also to the transient features of the input elements, the response acceleration of the masses are variable permanently. In these conditions, the maximum efforts will not be reached simultaneously in all cross sections.

5. Conclusive Remarks

As it was pointed out in [1], the study dealing with structures having uniformly mass variation over the height, when subjected to sinusoidal forced oscillations (transmitted through the supporting base), showed that for the fundamental vibration mode there appear differences between the instantaneous displacement shape derived in the classical solution and respectively the deflected one derived on behalf of propagation time delay of the oscillation in the case of flexible structures (with long natural period T_1) for large pulsations:

$$f_{exc} \geq \frac{3}{T_1}$$

By analogy with previous statement, we can extend this ascertainment to multistory buildings submitted to seismic actions, with lumped mass distribution. The phenomenon of „nonsynchronism“ of the oscillations will be significant when the structure of flexible type—that means with $T_1 \geq 1.5$ sec. — is situated in the neighbourhood of the seismic epicenters. For these regions the high frequency components of the seismic waves have a large weight, this being confirmed after major earthquakes that happened in these areas.

Concluding, we can appreciate that in the case of usual type structures, the well known equations of motion correspond approximately to the practical requirements. However, for high flexible structures, the equations of the type (2) lead to results closer to actual physical phenomenon of inducing the seismic vibrations to all levels of the multistory structure.

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UN PROCEDEU ȘI UN ALGORITM ÎN ANALIZA STRUCTURILOR MULTIETAJATE
ÎN FAZA TRANZITORIE A OSCILAȚIILOR SEISMICE

(Rezumat)

Se prezintă un algoritm de rezolvare a ecuației cunoscute de echilibru dinamic utilizând procedeul integrării pas cu pas în cazul când se ia în considerație nesincronismul oscilațiilor care se constată ca fenomen la structurile multietajate flexibile. Algoritmul elaborat corespunde domeniului liniar de comportare și poate fi extins și în domeniul neliniar.

POUTRES ENCASTRÉES À DEUX EXTRÉMITÉS, PRÉCONTRAINTES DANS LA TRAVÉE PAR UN TIRANT DISPOSÉ ASYMÉTRIQUEMENT

PAR

DAN PRECUPANU et RODICA BOAZU

1. Considérations générales. Hypothèses de calcul

Les poutres encastrées à leurs deux extrémités, précontraintes dans la travée par un tirant disposé asymétriquement, (Fig. 1, où t — tirant de précontrainte), sont utilisées comme des structures de résistance indépendantes, ou comme des systèmes de base pour le calcul des structures hyperstatiques. Leur dimensionnement, en employant la méthode de la section équivalente [1], implique la nécessité de connaître le moment fléchissant total repris par l'ensemble poutre + tirant de la section de calcul. Dans ce qui suit on présente les relations de calcul à cet effort.

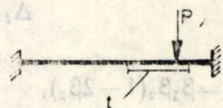


Fig. 1

On admet les hypothèses de calcul ci-dessous :

- a) La poutre est droite et a la section constante tout le long de celle-ci.
- b) Le matériel de la poutre et du tirant respecte les hypothèses du calcul linéaire élastique tant dans la phase de précontrainte que dans celle d'exploitation.
- c) Le tirant de précontrainte est rectiligne et parallèle à l'axe longitudinal de la poutre.
- d) La stabilité de la structure est assurée tant à la précontrainte du tirant que dans le stade d'exploitation.

2. Établissement des relations de calcul

2.1. Efforts produits par l'action de la force d'exploitation

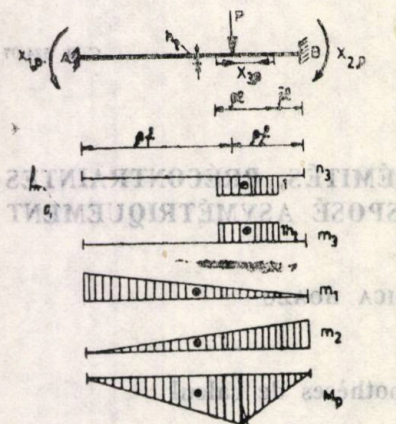
On analyse le cas du chargement avec une force concentrée appliquée à une distance quelconque, $\beta_1 l$, de l'extrémité A (Fig. 2). Le tirant est assemblé à la poutre sans être tendu.

On a noté par : $X_{1,p}$ — le moment d'encastrement de l'extrémité A de la barre ; $X_{2,p}$ — le moment d'encastrement de l'extrémité B de la

barre ; $X_{3,p}$ — l'effort du tirant qui représente d'ailleurs l'effort d'auto-tension. La condition de compatibilité des déplacements est :

$$(1) \quad [\delta][X_p] = \{\Delta_p\},$$

où :



$$(2) \quad [\delta] = \frac{1}{EI} \begin{bmatrix} \frac{l}{3} & \frac{l}{6} & a\beta lh_t \\ \frac{l}{6} & \frac{l}{3} & (1-a)\beta lh_t \\ -a\beta lh_t & (1-a)\beta lh_t & (h_t^2 + I/A)\beta l \end{bmatrix}$$

$$\{X_p\} = \begin{bmatrix} X_{1,p} \\ X_{2,p} \\ X_{3,p} \end{bmatrix}, \quad \{\Delta_p\} = \frac{1}{EI} \begin{bmatrix} A \frac{Pl^2}{6} \\ B \frac{Pl^2}{6} \\ Ch_t \frac{Pl^2}{2} + \bar{\Delta}_t \end{bmatrix};$$

Fig. 2

$$\bar{\Delta}_t = -\frac{X_{3,p} l_t}{E_t A_t} EI = -\frac{EI}{E_t A_t} \beta l X_3 = -\gamma \beta l X_3,$$

$$A = \beta_1 \beta_2 (1 - 2\beta_2), \quad B = \beta_1 \beta_2 (1 + 2\beta_1), \quad C = \beta_1 \beta_2 \beta + \beta_2 b (\beta_1 - b) + \beta_1 \beta (\beta_2 - \bar{\beta}),$$

$$a = \bar{\beta} + \frac{\beta}{2}, \quad b = 1 - \beta - \bar{\beta}, \quad \gamma = \frac{EI}{E_t A_t}.$$

Ici EI représente la rigidité de la section de la poutre et $E_t A_t$ — la rigidité du tirant. À l'aide de l'équation (1) on obtient :

$$X_{1,p} = \frac{Pl}{3} \cdot \frac{(h_t^2 + I/A + \gamma)(2A - B) + 6\beta(1-a)h_t^2[Ba - A(1-a)] + 3Ch_t^2(1-3a)}{h_t^2[1 + 4\beta(3a - 3a^2 - 1)] + I/A + \gamma},$$

$$X_{2,p} = \frac{Pl}{3} \cdot \frac{(h_t^2 + I/A + \gamma)(2B - A) + 6\beta ah_t^2[A(1-a) - Ba] + 3Ch_t^2(3a - 2)}{h_t^2[1 + 4\beta(3a - 3a^2 - 1)] + I/A + \gamma},$$

$$X_{3,p} = \frac{Pl}{3} \cdot \frac{(1-a)h_t(A - 2B) + ah_t(B - 2A) + 3Ch_t/2\beta}{h_t^2[1 + 4\beta(3a - 3a^2 - 1)] + I/A + \gamma}.$$

On peut écrire les relations précédentes sous la forme suivante :

$$(3) \quad X_{1,p} = \mathcal{M}_1 D_1, \quad (4) \quad X_{2,p} = \mathcal{M}_1 D_2, \quad (5) \quad X_{3,p} = \mathcal{M}_1 D_3,$$

où $\mathcal{M}_1 = P\beta_1\beta_2 l$ représente le moment d'encastrement de l'extrémité A de la barre non précontrainte (sans tirant) et les coefficients D_1, D_2, D_3 , sont donnés par les rapports ci-dessous :

$$(6) D_1 = \frac{1}{3\beta_1\beta_2^2} \cdot \frac{(h_t^2 + I/A + \gamma)(2A - B) + 6\beta(1-a)h_t^2[Ba - A(1-a)] + 3Ch_t^2(1-3a)}{h_t^2[1 + 4\beta(3a - 3a^2 - 1)] + I/A + \gamma},$$

$$(7) D_2 = \frac{1}{3\beta_1\beta_2^2} \cdot \frac{(h_t^2 + I/A + \gamma)(2B - A) + 6\beta ah_t^2[A(1-a) - Ba] + 3Ch_t^2(3a - 2)}{h_t^2[1 + 4\beta(3a - 3a^2 - 1)] + I/A + \gamma},$$

$$(8) D_3 = \frac{1}{3\beta_1\beta_2^2} \cdot \frac{(1-a)h_t(A - 2B) + ah_t(B - 2A) + 3Ch_t/2\beta}{h_t^2[1 + 4\beta(3a - 3a^2 - 1)] + I/A + \gamma}.$$

2.2. Efforts produits à la prétension du tirant

En appliquant au tirant l'effort X_0 , les encastresments A et B de la poutre sont soumis aux moments fléchissants $X_{1,0}$ et $X_{2,0}$, donnés par les relations :

$$(9) X_{1,0} = 2\beta h_t X_0(1 - 3a), \quad (10) X_{2,0} = 2\beta h_t X_0(3a - 2).$$

2.3. Efforts dans la phase d'exploitation

En superposant les effets des hypothèses de chargement étudiées dans les §§ 2.1 et 2.2, il résulte :

a) le moment de l'encastrement A :

$$(11) X_1 = X_{1,p} + X_{1,0};$$

b) le moment de l'encastrement B :

$$(12) X_2 = X_{2,p} + X_{2,0};$$

c) l'effort du tirant :

$$(13) X_3 = X_{3,p} + X_0.$$

L'effort total du tirant est $X_3 = A_{te}R$, ce qu'on peut écrire sous la forme $X_3 = A_{te}R$, où $A_{te} = \alpha A_t$ et $\alpha = R_t/R$ (R — la résistance de l'acier du tirant, R — la résistance de la poutre au niveau du tirant).

À l'aide des relations (13), (5) on obtient :

$$X_0 = X_3 - X_{3,p} = A_{te}R - \mathcal{M}_1 D_3.$$

En introduisant successivement dans (9), (10) et puis dans (11), (12) il résulte les relations :

$$(14) X_1 = \mathcal{M}_1 D_1 + 2\beta h_t(1 - 3a)(A_{te}R - \mathcal{M}_1 D_3),$$

$$(15) X_2 = \mathcal{M}_1 D_2 + 2\beta h_t(3a - 2)(A_{te}R - \mathcal{M}_1 D_3).$$

Le rapport

$$(16) c_{M_1} = \frac{X_1}{\mathcal{M}_1},$$

que l'on peut nommer le coefficient de correction du moment de l'encastre-

ment A , représente le facteur avec lequel on doit multiplier le moment de l'encastrement de la barre non précontrainte.

En introduisant la notation (16) en (14) et compte tenu de la condition de résistance de la section équivalente, $X_1/W_{ze} = R$, on peut écrire :

$$c_{M_1} \mathcal{M}_1 = \mathcal{M}_1 D_1 + 2\beta h_t (1 - 3a) \left(A_{te} \frac{c_{M_1} \mathcal{M}_1}{W_{ze}} - \mathcal{M}_1 D_3 \right),$$

d'où il résulte :

$$(17) \quad c_{M_1} = \frac{D_1 - 2\beta h_t (1 - 3a) D_3}{1 - 2\beta h_t (1 - 3a) A_{te} / W_{ze}},$$

Tous calculs effectués, on constate que :

$$D_1 - 2\beta h_t (1 - 3a) D_3 = \frac{2A - B}{3\beta_1 \beta_2^2}$$

et par conséquent

$$c_{M_1} = \frac{(2A - B) / 3\beta_1 \beta_2^2}{1 - 2\beta h_t (1 - 3a) A_{te} / W_{ze}}.$$

D'une façon analogue, en notant par :

$$(18) \quad c_{M_2} = \frac{X_2}{\mathcal{M}_1}$$

la relation (15) devient :

$$c_{M_2} \mathcal{M}_1 = \mathcal{M}_1 D_2 + 2\beta h_t (3a - 2) \left(A_{te} \frac{c_{M_2} \mathcal{M}_1}{W_{ze}} - \mathcal{M}_1 D_3 \right)$$

d'où on obtient :

$$c_{M_2} = 2\beta h_t (3a - 2) A_{te} \frac{c_{M_2}}{W_{ze}} + D_2 - 2\beta h_t (3a - 2) D_3.$$

Mais :

$$D_2 - 2\beta h_t (3a - 2) D_3 = \frac{1}{3\beta_1 \beta_2^2} (2B - A) = \frac{1}{3} (1 + 2\beta_1 - \beta_2)$$

et par conséquent :

$$(19) \quad c_{M_2} = \frac{1}{3\beta_1 \beta_2^2} (1 + 2\beta_1 - \beta_2) + 2\beta h_t (3a - 2) \frac{A_{te}}{W_{ze}} c_{M_1}.$$

3. La discussion des facteurs de correction

Si l'on analyse des relations de calcul (17), (19), on constate que les facteurs de correction c_{M_1} , c_{M_2} dépendent du rapport A_{te}/W_{ze} et de la position du tirant par rapport à la poutre (β , $\bar{\beta}$, h_t). En ce qui concerne l'évaluation de ces paramètres, on peut faire les observations suivantes :

a) si le tirant est fixé au niveau de la face de la poutre, le terme $h_t A_{te}/W_{ze}$ dépend exclusivement du degré de précontrainte de la poutre (A_{te}/W_{ze}) [2]; b) les valeurs des facteurs de correction sont invariables par rapport à la forme de la section et par conséquent on peut les calculer en considérant une section de forme arbitraire, par exemple en double T.

Dans le Tableau 1 sont donnés les facteurs c_{M_1} , c_{M_2} pour la poutre représentée dans la Fig. 2.

4. Conclusions

Pour les poutres doublement encastrées, précontraintes en travée, le facteur de correction c_{M_1} est sousunitaire. Sa valeur diminue d'autant plus que le degré de précontrainte est plus élevé; pour les poutres précontraintes à $A_{te}/A_{se}=1,00$, ce qui signifie que le tirant remplace totalement la semelle tendue du profil équivalent, le facteur c_{M_1} atteint des valeurs comprises entre 0,75 ... 0,95.

Si le degré de précontrainte ne dépasse pas 50% ($A_{te}/A_{se} < 0,5$) la différence entre la valeur du moment d'encastrement X_1 de la poutre précontrainte et celle de la poutre non précontrainte (sans tirant) est inférieure à 5%.

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GRINZI DUBLU ÎNCASTRATE, PRETENSIONATE ÎN DESCHIDERE CU UN TIRANT DISPUS ASIMETRIC

(Rezumat)

Se analizează problema determinării momentelor de încadrare perfectă la aceste tipuri de bare, încărcate cu o forță concentrată. Se ajunge la concluzia că momentul de încadrare perfectă din grinda pretensionată se poate determina cu relația $X_1 = c_{M_1} M_1$, în care M_1 reprezintă momentul de încadrare perfectă calculat pe bara nepretensionată (fără tirant) iar c_{M_1} , un factor de corecție care depinde de gradul de pretensionare, de excentricitatea tirantului și de poziția punctului de aplicație al forței exterioare în lungul grinzii.

Se prezintă tabele cu valori numerice ale factorilor c_M pentru unele cazuri particulare.

VÉRIFICATION À LA FATIGUE DES ASSEMBLAGES PAR VIS À HAUTE RÉSISTANCE, PRÉCONTRAINTS

PAR

C. ȘERBESCU

1. Introduction

Dans ce qui suit on présente le calcul à la fatigue de certains assemblages par vis à haute résistance, précontraints. On analyse le cas des assemblages où les efforts d'intensité variable dans le temps produisent des contraintes de tension dans la tige des vis; le calcul est effectué conformément à la norme technique roumaine 10 108/0-78.

2. Verification à la fatigue

On vérifie le phénomène de la fatigue des éléments et des assemblages actionnés par des efforts variables (répétables à des intervalles réduites de temps dans un grand nombre), tels que : les poutres des voies de roulement, des plate-formes des voies et des éléments des estacades actionnés par des charges mobiles. L'apparition du phénomène de fatigue pour un coefficient d'asymétrie $\sigma_{\min}/\sigma_{\max}$ connu (Fig. 1 a) dépend du nombre

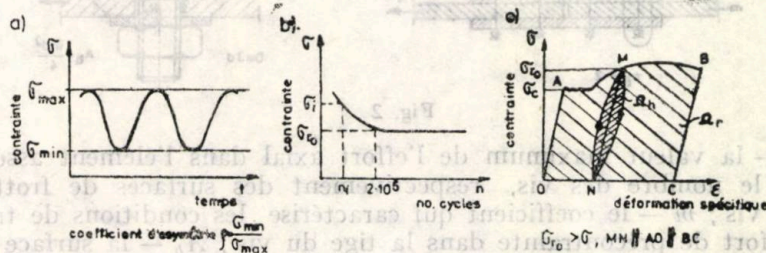


Fig. 1

de cycles, n (Fig. 1 b); pour les poutres de soutènement des voies de roulement on considère que le phénomène de la fatigue (la contrainte σ_{r0}) peut apparaître pour $n=0,5 \dots 2$ millions de cycles.

La détermination du nombre des cycles (n) au quel se produit la rupture de l'éprouvette ($\sigma_{\max}=\sigma_{r0}$) résulte par l'essai à la fatigue ou à la

traction (Fig. 1 c) en fonction de la caractéristique de résistance (Ω_r) et d'hystérésis (Ω_h), à savoir : $n = \Omega_r / \Omega_h$.

La vérification à la fatigue est réalisée à l'aide de la relation :

$$(1) \quad \sigma_{\max} \leq \gamma R,$$

où :

$$\gamma = \frac{c}{a - b\rho}, \quad \rho = \frac{\sigma_{\min}}{\sigma_{\max}}, \quad \sigma_{\min} > 0.$$

Ici a , b , c dépendent du nombre des cycles de chargement dans la période de l'exploitation de la construction (n); pour la marque d'acier OL 37 $a=1,55$, $b=0,95$, $c=1$ (pour $n=2 \cdot 10^6$), $c=1,1$ (pour $n=10^6$), $c=1,3$ (pour $n=0,5 \cdot 10^6$).

Les assemblages réalisés avec des vis à haute résistance précontraints, travaillent d'une façon différente par rapport aux celles avec des vis ordinaires, le transfert des efforts axiaux étant réalisé par le frottement entre les surfaces des éléments en contact.

La valeur élevée de la force nécessite un grand effort de précontrainte dans la tige du vis, ce qui impose que l'acier du vis ait une limite de fluage élevée (par traitements thermiques et alliages). Le serrage du vis pour l'obtention de la force de précontrainte nécessaire (N) est réalisée avec des clés dynamométriques.

On effectue la vérification d'un assemblage par vis à haute résistance précontraints (Fig. 2) à l'aide de la relation :

$$(2) \quad N \leq n n_F m N_t f = 0,8 n n_F m A_b R_c f,$$

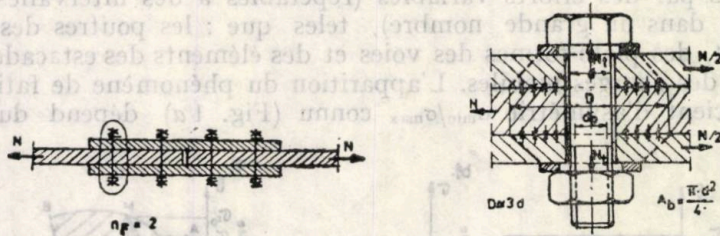


Fig. 2

où : N — la valeur maximum de l'effort axial dans l'élément assemblé ; n , n_F — le nombre des vis, respectivement des surfaces de frottement pour un vis ; m — le coefficient qui caractérise les conditions de travail ; N — l'effort de précontrainte dans la tige du vis ; A_b — la surface de résistance du filet du vis ; R — la limite de fluage minimum de l'acier ($R = 6\,400 \dots 10\,800 \text{ daN/cm}^2$) ; f — le coefficient du frottement (pour OL 37, $f=0,40$).

Si le vis est actionné, aussi, par un effort de tension dans la tige, Z , la relation (2) prend la forme :

$$(3) \quad N \leq n n_F m (N_t - Z) f,$$

où, pour des efforts d'intensité variable dans le temps, $Z \leq 0,6 N_t$.

Pour les assemblages avec des vis à haute résistance, précontraints, actionnés par les forces N et Z (Fig. 3), on remarque deux phases de travail : la première phase — pour la force de précontrainte N et la deuxième — pour la force de précontrainte N et les forces extérieures Z .

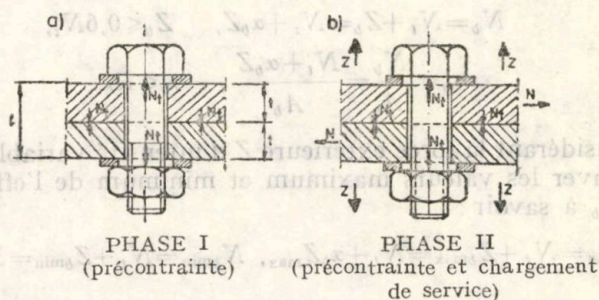


Fig. 3

Dans la première phase (Fig. 3 a), par suite à la précontrainte, la longueur de la tige du vis croît de l à $l + \Delta l_1$ et l'épaisseur des éléments assemblés est réduite de t à $t - \Delta t_1$.

Dans la deuxième phase (Fig. 3 b), par suite à l'application de la force extérieure Z , la longueur de la tige du vis croît avec Δl_2 et l'épaisseur des éléments assemblés croît avec Δt_2 (de tel façon que $\Delta l_2 = \Delta t_2$). La force Z produit des forces de tension dans la tige du vis (Z_p) et dans les éléments assemblés (Z_b) de sorte que :

$$(4) \quad Z = Z_p + Z_b.$$

En imposant la condition :

$$(5) \quad \Delta t_2 = \Delta l_2,$$

on obtient pour les deux termes :

$$(6) \quad \frac{Z_p l_p}{E_p A_p} = \frac{Z_b l_b}{E_b A_b}, \quad \frac{Z_p}{K_p} = \frac{Z_b}{K_b},$$

où :

$$K_p = \frac{E_p A_p}{l_p}, \quad K_b = \frac{E_b A_b}{l_b}.$$

Des relations (4) et (6) il résulte :

$$(7) \quad \frac{Z_p}{K_p} = \frac{Z_b}{K_b} = \frac{Z_p + Z_b}{K_p + K_b} = \frac{Z}{K_p + K_b}, \quad Z_p = \alpha_p Z, \quad Z_b = \alpha_b Z,$$

où :

$$\alpha_p = \frac{K_p}{K_p + K_b}, \quad \alpha_b = \frac{K_b}{K_p + K_b}.$$

Si l'on tient compte que :

$$E_p = E_b = E, \quad l_p = l_b = l,$$

$$A_b = \frac{\pi d^2}{4}, \quad A_p = \frac{\pi D^2}{4} - \frac{\pi d^2}{4} = 2\pi d^2 = 8A_b,$$

il résulte :

$$K_p = 8K_b, \alpha_p = 0,89, \alpha_b = 0,11.$$

L'effort N_b et la contrainte σ_b dans la tige du vis sont donnés par les relations :

$$(8) \quad N_b = N_t + Z_b = N_t + \alpha_b Z, \quad Z_b \leq 0,6 N_t,$$

$$(9) \quad \sigma_b = \frac{N_b}{A_b} = \frac{N_t + \alpha_b Z}{A_b}, \quad \sigma_b \leq R_b.$$

En considérant la force extérieure Z d'intensité variable dans le temps, on peut trouver les valeurs maximum et minimum de l'effort N_b et de la contrainte σ_b à savoir :

$$(10) \quad N_{b\max} = N_t + Z_{b\max} = N_t + \alpha_b Z_{\max}, \quad N_{b\min} = N_t + Z_{b\min} = N_t + \alpha_b Z_{\min},$$

et

$$(11) \quad \sigma_{b\max} = \frac{N_{b\max}}{A_b} = \frac{N_t + \alpha_b Z_{\max}}{A_b}, \quad \sigma_{b\min} = \frac{N_{b\min}}{A_b} = \frac{N_t + \alpha_b Z_{\min}}{A_b}.$$

À l'aide de la relation (1) on peut vérifier que :

$$(12) \quad \sigma_{b\max} = \frac{N_{b\max}}{A_b} = \frac{N_t + \alpha_b Z_{\max}}{A_b} \leq \gamma R_c.$$

Par exemple, pour l'acier OL 37 et $n=2.10^6$, on obtient :

$$(13) \quad a=1,55, \quad b=0,95, \quad c=1; \quad \gamma = \frac{1}{1,55-0,95\rho} \approx \frac{0,64}{1-0,61\rho};$$

$$\rho = \frac{\sigma_{b\min}}{\sigma_{b\max}} = \frac{N_t + \alpha_b Z_{\min}}{N_t + \alpha_b Z_{\max}}.$$

Les assemblages poutre — poteaux, réalisés par des vis à haute résistance, précontraints, dans le cas des forces dont l'intensité est variable dans le temps (Fig. 4), impliquent la nécessité de recourir à des vérifications à l'aide des relations :

$$(14) \quad T \leq n n_F m N_t f = 0,8 n n_F m A_b R_c f,$$

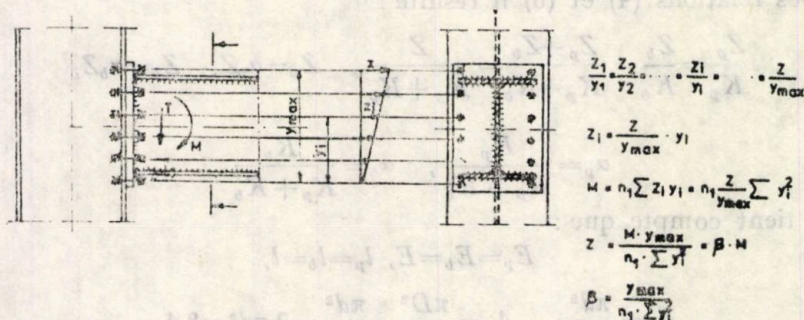


Fig. 4

(pour le cas de la Fig. 4, $n=10$, $n_F=1$, $m=0,9$, $f=0,40$) et donc (v. (10)),

$$(15) \quad N_{b\max} = N_t + \alpha_b \beta M_{\max}, \quad N_{b\min} = N_t + \alpha_b \beta M_{\min},$$

où

$$\beta = \frac{y_{\max}}{n_1 \sum_i y_i^2}, \quad Z_{b\max} \leq 0,6 N_t,$$

respectivement (v. (11)) :

$$(16) \quad \sigma_{b\max} = \frac{N_t + \alpha_b \beta M_{\max}}{A_b} \leq \gamma R_c, \quad \sigma_{b\min} = \frac{N_t + \alpha_b \beta M_{\min}}{A_b},$$

et

$$\rho = \frac{\sigma_{b\min}}{\sigma_{b\max}} = \frac{N_t + \alpha_b \beta M_{\min}}{N_t + \alpha_b \beta M_{\max}}.$$

3. Conclusions

Le calcul à la fatigue des assemblages par vis à haute résistance, précontraints, dépend de la façon dont actionnent les efforts à savoir :

a) pour le cas de la Fig. 3 b :

$$\sigma_{b\max} = \frac{N_t + \alpha_b Z_{\max}}{A_b} \leq \gamma R_c, \quad \rho = \frac{N_t + \alpha_b Z_{\min}}{N_t + \alpha_b Z_{\max}};$$

b) pour le cas de la Fig. 4 :

$$\sigma_{b\max} = \frac{N_t + \alpha_b \beta M_{\max}}{A_b} \leq \gamma R_c, \quad \rho = \frac{N_t + \alpha_b \beta M_{\min}}{N_t + \alpha_b \beta M_{\max}},$$

où le coefficient α_b tient compte des rigidités des éléments des assemblages et β dépend de la distribution des vis dans ceux-ci.

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VERIFICAREA LA OBOSEALĂ A UNOR ÎMBINĂRI CU ȘURUBURI DE ÎNALTĂ REZISTENȚĂ, PRETENSIONATE

(Rezumat)

Se prezintă un studiu al verificării la oboseală a unor îmbinări cu șuruburi de înaltă rezistență, pretensionate, la care încărcările exterioare dau naștere la eforturi unitare de întindere în tijă. Sînt analizate două cazuri de îmbinări, prezentîndu-se și relațiile de calcul respective.

STUDIUM BEZÜGLICH DER EFFIZIENZ DER ENERGIEKONSERVIERUNGSMASSNAHMEN UND VERWERTUNG DER SONNENENERGIE FÜR EIN EXPERIMENTELLES GEBÄUDE IN JASSY

VON

RODICA SARDINO und C. LEPĂDATU

1. Einführung

Die Wohn-, Industrie- und landwirtschaftlichen Bauten nehmen am nationalen Energieverbrauch mit einer bedeutenden Quote teil, da sie die Heizung der Innenräume im Winter und die Wassererwärmung zeits der ganzen Nutzdauer benötigen. Es ist also sehr wichtig, daß durch die Anwendung einer funktionellen, baulichen und installationstechnischen Maßnahmen dieser Verbrauch auf ein Minima herabgesetzt werden soll. Solche Bemühungen sind in der ganzen Welt sehr aktuell und verfolgen das Anwachsen der Energiekonservierungskapazität der Bauten, gleichzeitig mit der Verwertung einer neuen Energiequellen.

Die Energiekonservierungsmaßnahmen sind prinzipiell folgende:

- a) Verwendung der natürlichen Schutzelemente (Bepflanzungen, Bodengefälle).
 - b) Verbesserung des Wärmeschutzgrades der vollen Aussenlemente (im laufenden Feld und in den kritischen Zonen) und der Verglasungen der Innenelemente zwischen Räumen unterschiedlicher Temperatur.
 - c) Herabsetzung der Wärmeverluste durch Undichtigkeiten.
 - d) Verwendung rationaler Oberflächen für die Fenster der Räume.
 - e) Realisierung minimaler Aussenflächen durch Anwendung effizienter Formen für das Gebäude.
 - f) Bewerkstellung von Pufferzonen zwischen den Wohnräumen und Norden.
- Unter der unkonventionellen Energiequellen, nimmt die Sonnenenergie einen Vorzugsplatz ein, wobei ihre Verwertung durch folgende Maßnahmen möglich wird:
- a) Einfangen, Stapelung der Sonnenenergie und Heizung der Räume im aktiven System.
 - b) Einfangen, Stapelung der Sonnenenergie und Heizung der Räume im passiven System.
 - c) Einfangen und Sammlung der Sonnenenergie zwecks Warmwasseraufbereitung für Verbrauch, jahreszeitenabhängig oder ganzjährig.
 - d) Wärmewiedergewinnung aus der Abluft mit Sonnenaktivierung.

Für die zweckmässige Verwertung der Sonnenenergie ist es notwendig, daß das Gebäude Südorientierung haben soll (die Abweichungen von dieser Orientierung sollen 30° nicht überschreiten) und die Sonnenstrahlung in der kalten Jahreszeit soll nicht durch Bäume oder andere Gebäude verhindert sein.

Sowohl in unserem Land als auch im Ausland gibt es zahlreiche Bemühungen und Lösungen bezüglich der Wasserwärmung und Gebäudenheizung durch Verwertung der Sonnenenergie.

Im Innland wurde die Sonnenenergie vor allem für die Wasserwärmung für den Warmwasserverbrauch verwertet.

Nur einige Gebäude verwenden die Sonnenenergie auch für die Heizung der Räume im aktiven System, im passiven oder kombinierten (die beiden Sonnenhäuser aus Cimpina CS1, CS2; das Sonnenhaus von INCERC, Bukarest, CS3; die vier Wohnbauten P+2 Stockwerke aus Medgidia).

Für die Anwendung und Erweiterung der Systeme der Energiekonservierung und Verwertung der Sonnenenergie in den nachteiligen Gegebenheiten der Stadt und des Bezirks Jassy (und im allgemeinen des nördlichen Teils des Landes) ist ein Dauertest eines Gebäudes in Funktion notwendig. Nur durch theoretische Analysen und begrenzte Versuche, im Labor, sind die Resultate nicht eindeutig um im Entwurf und Ausführung übernommen zu werden.

Darum wird ein solches experimentelles Wohngebäude im Stadtviertel Zugravi der Munizipiums Jassy gebaut (Nutzniesser und Investitionsausführung—T.A.G.C. Ind., Jassy).

Für dieses Wohngebäude wird im folgenden die Resultate der ausgeführten Forschungen bezüglich der funktionellen Lösung, der prinzipiellen Baugestaltung, der Systeme und der spezifischen Installationen für die Verwertung der Sonnenenergie, vorgestellt, um dem Gebäude einen hohen energetischen Autonomiegrad zu verleihen¹⁾.

2. Gebäudebeschreibung

Das Gebäude besitzt P+3 Stockwerke, so wie auch die anderen Wohnbauten aus dem Stadtviertel, entworfen von ICPROM, Jassy, ist aber als ein bioklimatisches Gebäude gedacht, und weicht so von vorhergehenden durch die Architektur, funktionelle Lösung, spezifische Lösungen bezüglich des Anwachsens des Energiekonservierungsgrades und Verwertung der Sonnenenergie ab. Die Tragstruktur besteht aus Betonscheiben (abweichend von den anderen Gebäuden des Stadtviertels, bei welchen man Tragwände aus Ziegelmauerwerk verwendete). Ausgehend von der Idee des Entwurfs in der Zukunft für eine große Gebäudeanzahl, beantwortete man die Forderung des Nutzniessers der Forschungsarbeiten und der aktuellen Tendenz im Baubereich, bezüglich des Anwachsens des Industrialisierungsgrades und der Baustellenproduktivität.

Die mässige Höhe wurde nicht nur durch gute Eingliederung im Stadtviertel sondern auch von der Möglichkeit bestimmt, daß man auf Grund der erhaltenen Resultate in der Funktion des Testgebäudes ein Typ-Entwurf für Wohnbauten mit hohem energetischen Autonomiegrad für Stadt- und Landwohnungen ausarbeiten soll. Die Realisierung solcher Gebäude in den Dörfern führt gleichzeitig zur Befriedigung der Systematisierung der Ortschaften, zur Herabsetzung des Energieverbrauchs für Heizung der Wohnungen und Warmwasseraufbereitung und an die Annäherung bezüglich des Konforts an die Wohnbauten aus den Städten.

¹⁾ Die Forschungsarbeiten wurden an der Fakultät für Bauwesen Jassy, Lehrstuhl für Bauwerke (funktionelle Lösung, bauliche Lösungen, Verwertung der Sonnenenergie für Heizung im passiven System) und durch ICPROM (Verwertung der Sonnenenergie im aktiven System und Warmwasseraufbereitung) durchgeführt.

Das Gebäude besitzt je 2 Wohnungen am Treppenhaus: 3 Wohnungen mit je 2 Zimmern, 2 Wohnungen mit je 3 Zimmern und 3 Wohnungen mit je 4 Zimmern.

Man gewährte eine große Aufmerksamkeit der Prinzipien aus § 1, bezüglich der Energiekonservierungsmaßnahmen und der Verwertung der Sonnenenergie.

Der Baugrund des Gebäudes, an einem Abhang mit einer S—O-Orientierung ermöglicht diesem eine gute Orientierung, südwärts und Schutz gegen dominierende Winde.

Bei der Annahme der funktionellen Lösung waren die inbetrachtgenommenen Kriterien die optimale Verbindung zwischen den Räumen, Einhaltung der Vorschriften bezüglich deren Oberfläche, aber auch das Kriterium auf Grund welchem, für das Anwachsen der Energieautonomiegrades, ist es nötig gegen Norden Pufferräume herzustellen, welche die Hauptwohnräume schützen sollen.

So also wurden die Wohnzimmer gegen Süden orientiert (Living, Schlafzimmer). Je ein Schlafzimmer einer Wohnung ist gegen Osten oder Westen orientiert. Der Pufferaum gegen Norden sind das Treppenhaus, Badezimmer, Speisekammern, Küchen. Die hauptsächlich Wohnräume sind ebenfalls gegen die kältere Zone des Treppenhauses durch einen Pufferaum aus Vorzimmer und Badezimmer bestehend (Abb. 1), geschützt.

Um die Wärmeverluste so klein wie möglich zu halten, wurden die Aussenflächen ebenfalls so klein wie möglich gehalten.

Im laufenden Feld wurden für die vollen Aussenbauelemente ein höherer Wärmeschutzgrad vorgesehen sowie auch für Innenelemente, welche Räume verschiedener Temperaturen abschließen (Aussenwände, Wände gegen das Treppenhaus, Aufbodendecke, Kellerdecke). Man verfolgte ebenfalls die thermischen Brücken so klein wie möglich zu halten, sodaß man für das Ganze der Bauelemente höhere thermische Widerstände erhielt als jene laut aktuellen Normen aus unserem Land für gewöhnliche Wohnbauten.

Für die Wände der Nordfassade, der Stockwerke 1, 2, 3, erhielt man einen Zuschuß thermischen Schutzes durch Herstellen von Mansarden, welche auch als Nutzräume benutzt werden können.

Für die Wände mit Orientierung nach S, O, W vergrößert der Glashauseffekt den thermischen Schutzgrad jenes Abschnittes der Fassade.

Die Überdachung mit Dachstuhl und Aufboden (Abb. 2) bewirkt durch die Luftschichte des Aufbodens die Verbesserung des thermischen Schutzes des oberen Teils des Gebäudes.

Durch dreifache Verglasung der Fenster und der Balkontüren und zweckmäßige Abdichtungsmaßnahmen (einschließlich der Eingangstüre der Wohnung) werden die Wärmeverluste verkleinert (welche eine große Wichte in der Struktur der Wärmeverluste des ganzen Gebäudes einnehmen).

Die Fenster der geheizten Räume wurden in ihrer Mehrzahl in die Südfassade verteilt (um den Sonnenanteil in der kalten Jahreszeit zu vergrößern und zur Verkleinerung dessen in der warmen Jahreszeit) u. zw. : a) Südfassade 56,7% ; b) Ost(west)fassade 19,8% ; c) Nordfassade 3,7%.

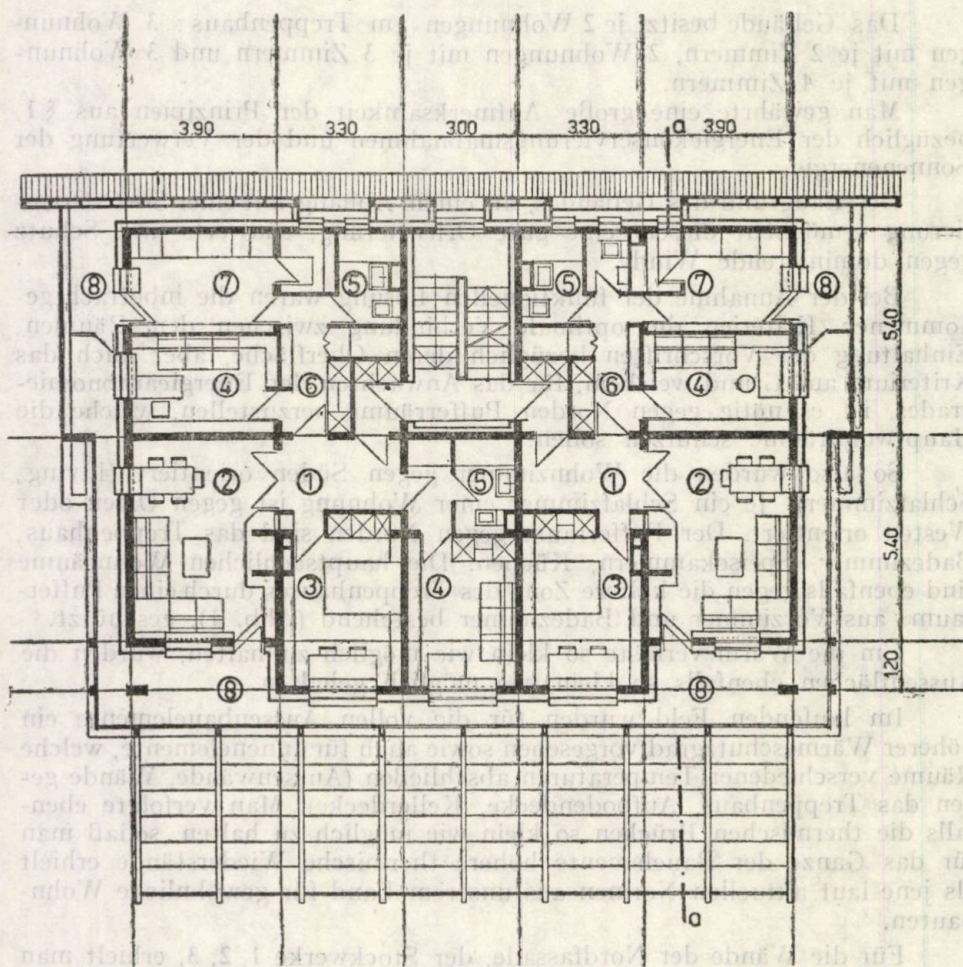


Abb. 1. — Obergeschoß 1: 1—Diele, 2—Wohnen, 3—Eltern, 4—Kinderzimmer, 5—Bad+WC
6—Flur, 7—Küche, 8—Glashaus.

Man zog die Verwertung der Sonnenenergie sowohl für die Heizung im passiven und aktiven System in der kalten als auch für die Erhaltung des Warmwassers für den Verbrauch in der warmen Jahreszeit hinzu.

Für die passive Heizung verwendete man das Glashaussystem. Das Glashaus wurde nicht nur an der Südfassade sondern auch an der Ost- und Westfassade vorgesehen, welchen eine kleinere Sonnenstrahlungsmenge in der kalten Jahreszeit entspricht, unter Berücksichtigung das der Glashaussystem zum Anwachsen des thermischen Schutzes des Gebäudes führt (dieser ist nicht durch die Anwesenheit des Sonnenanteils und die Orientierung der Fassade bestimmt).

Ein beweglicher thermischer Schutz der Verglasung verkleinert die Wärmeverluste in der kalten Jahreszeit und ermöglicht die Beschattung

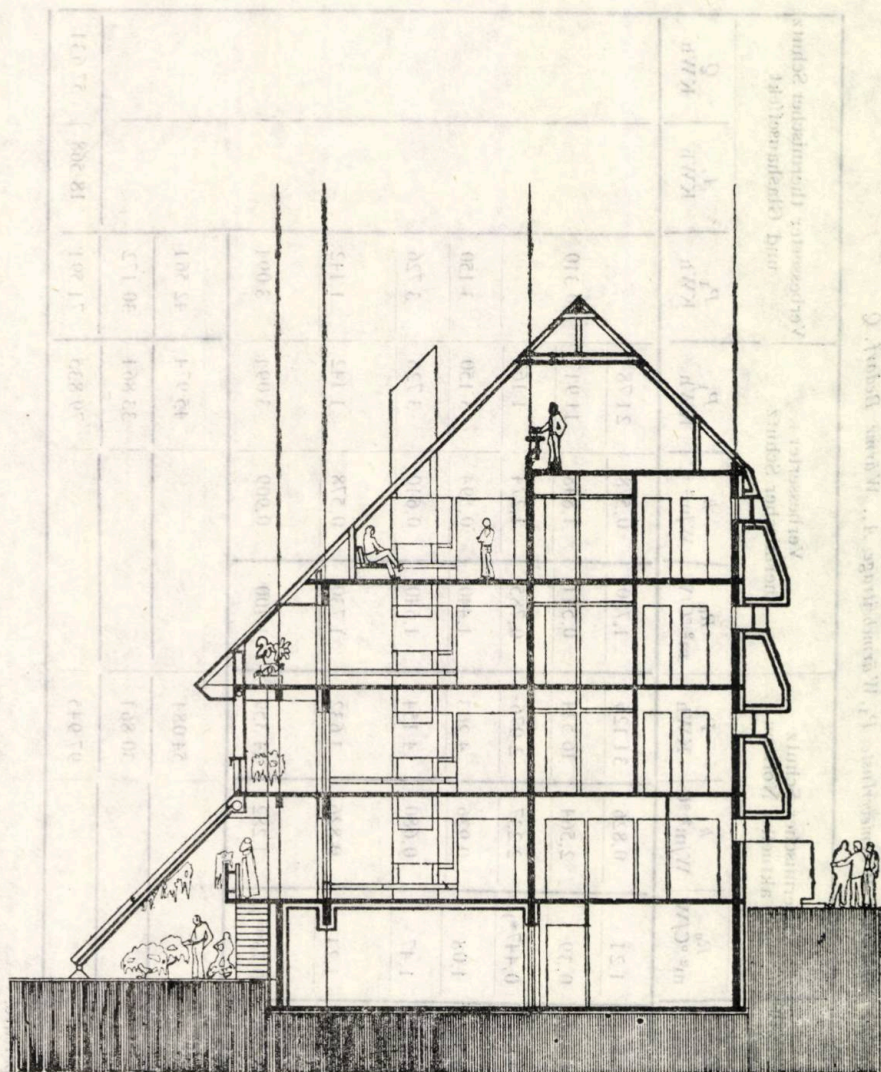


Abb. 2. — Schnitt *aa*.

1	Дубовый лес	0 483	
2	Лесной парк	0 780	
3	Полоса леса	11 540	
4	Сенокос	20 160	
5	Пашня	20 160	
6	Водоем	20 160	

Abb. 2.—Schnitt aa.

Tabelle 1

Struktur der jährlichen Wärmeverluste P_1 , Wärmebeiträge A_s , Wärme Bedarf, Q

Nr.	Element	Oberfläche m^2	Jahreszahl Grad/ Stunden $^{\circ}C \cdot h$	Thermischer Schutz laut aktueller Normen			Verbesselter thermischer Schutz			Verbesselter thermischer Schutz und Glashauseffekt		
				R_0 $m^2 \cdot ^{\circ}C/W$	k $W/m^2 \cdot ^{\circ}C$	P_1 KWh	R_0 $m^2 \cdot ^{\circ}C/W$	k $W/m^2 \cdot ^{\circ}C$	P_1 KWh	P_1 KWh	A_s KWh	Q KWh
1	Aussenwände	503,496	74 842	1,21	0,826	31 126	1,730	0,578	21 781			
2	Fenster	86,160	74 842	0,39	2,564	16 534	0,541	1,848	11 917	30 310		
3	Balkontüren	12,240	74 842	0,447*)	2,237	2 076	0,785	1,274	1 167			
4	Kellerdecke	117,700	38 592	1,08	0,926	4 203	1,440	0,594	3 150			
5	Aufbodendecke	93,386	65 410	1,47	0,680	4 154	1,640	0,610	3 726			
6	Mansardewände	26,400	74 842	1,21	0,826	1 632	1,730	0,578	1 142			
7	Wände des Treppenhauses	70,488	48 240	0,78**)	1,282	4 359	1,100	0,909	3 091			
Total Bauelemente						54 084			45 974	42 561		
Ventilatur						30 861			33 861	30 172		
Total Gebäude						97 945			79 835	71 591	18 568	57 631

*) Effektivwert für Doppeltüre.

**) Normwert für Wand bei geschlossener Fuge DCS 256/1984.

in der warmen Jahreszeit, zwecks Verhütung der Überwärmung des Glashauses und dieser Weise also der anliegenden Räume. Ebenfalls zwecks der Überwärmung in der warmen Jahreszeit, ermöglichen bewegliche Glashaufenster eine gute Belüftung.

Man untersuchte auch die Möglichkeit der Heizung im aktiven System, mit Warmluft, mit täglicher Sammlung. Zu diesem Zweck können auf das Dach Sonnenkaptoren mit Luft montiert werden. Die Luft welche im Sammler erwärmt wurde, kommt in eine Sammelzone im Keller und wird dann verwendet, wenn die Sonnenstrahlung fehlt.

Zwecks des Erhaltens des warmen Wassers für den Verbrauch in der warmen Jahreszeit, sind auf dem Dach Sonnensammler mit Wasser montiert.

Für die Versorgung in den Stoßzeiten mit Wärme wurde auch eine klassische Heizung vorgesehen.

3. Effizienz der Energiekonservierungsmassnahmen und der Verwertung der Sonnenenergie

Das übliche Kriterium für die Bewertung vom thermischen Gesichtspunkt aus der Bauelemente, welche Räume verschiedener Temperaturen abschliessen, ist der thermische Widerstand, R_0 . Bei der Bestimmung der Normwerte dieses Kriteriums, $R_{0\text{ nec}}$, muß man die gleichzeitige Erfüllung des hygrothermischen Konforts und die Verkleinerung der Investitions- und Gebrauchskosten (durch die Verkleinerung des Energieverbrauchs für Heizung) in Betracht ziehen. Das Bestreben der Verbesserung der Normwerte dieses Kennwertes führte zu dessen Abänderung in der Zeit, sowohl in unserem Land als auch im Ausland.

Für die Beurteilung des thermischen Verhaltens des gesamten Gebäudes, kann man die Wärmeverluste zeits der ganzen Heizungszeit, P , in Betracht nehmen.

In Tabelle 1 sind die jährlichen Wärmeverluste für das untersuchte Gebäude in zwei Hypothesen angegeben: thermischer Schutz der Bauelemente laut gültigen Vorschriften aus unserem Land und ein verbesserter thermischer Schutz. Es folgt eine Verkleinerung der jährlichen Wärmeverluste von 97 945 KWh auf 79 835 KWh.

Die Realisierung der Glashäuser durch die Verglasung der Fassaden ermöglicht die Herabsetzung der Wärmeverluste durch Addition zwei verschiedener Effekte: Anwachsen des thermischen Schutzgrades der betreffenden Fassadenabschnitten und durch den Glashauseffekt.

Die Realisierung des verbesserten thermischen Schutzes und die Verwertung der Sonnenenergie im passiven System durch Glashäuser verkleinert auf 57 631 KWh die jährlichen Wärmeverluste, was zu einer Einsparung von 4 952 Kg konventionellem Heizstoff (s. Tabelle 1) führt.

Durch die Verwertung der Sonnenenergie für die Heizung im aktiven System mit täglicher Ansammlung (Sammelfläche 200 m², Sammelvolumne 85 m³) und für die Aufbereitung des Warmwassers für den Verbrauch

(Sammelfläche 32 m², Sammelbehälterfassungsvermögen 1 600 l) erhält man eine jährliche Einsparung konventioneller Heizstoffe von 2 377 Kg und bzw. 2 560 Kg.

4. Schlussfolgerungen

Das Bestreben bezüglich der Verkleinerung des Energieverbrauchs der Gebäude in Funktion ist in der ganzen Welt sowie auch in unserem Land sehr aktuell. Dieses Bestreben konkretisierte sich durch das Erstellen von Gebäuden mit einem höheren energetischen Autonomiegrad, auf Grund der zweckmäßigen Maßnahmen der Energiekonservierung und der Verwertung der Sonnenenergie.

Aus den Forschungen der Verfasser folgte, daß auch in den nachteiligen klimatischen Bedingungen des Bezirks und der Stadt Jassy, die Anwendung dieser Maßnahmen zu großen Einsparungen konventioneller Heizstoffe für die Heizung und Warmwasseraufbereitung führen kann.

Die Resultate der theoretischen Untersuchungen werden weiterhin am Gebäude in Funktion verfolgt werden.

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Lehrstuhl für Bauwerke
und
ICPROM, Jassy

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STUDIU PRIVIND EFICIENȚA MĂSURILOR DE CONSERVARE A ENERGIEI ȘI VALORIFICARE A ENERGIEI SOLARE PENTRU O CLĂDIRE EXPERIMENTALĂ ÎN IAȘI

(Rezumat)

Măsurile corespunzătoare de conservare a energiei și valorificare a energiei solare determină reducerea consumului de energie în exploatarea clădirilor.

Din cercetările efectuate de autori a rezultat că și în condițiile climatice defavorabile ale județului și orașului Iași, aplicarea acestor măsuri determină economii notabile de combustibil convențional pentru încălzire și prepararea apei calde de consum.

LA CAPACITÉ PORTANTE DES ÉLÉMENTS MÉTALLIQUES FLÉCHIS ENDOMMAGÉS PAR LA CORROSION

PAR

C. ȘERBESCU

1. Introduction

La sécurité d'une construction métallique dépend de la qualité du projet, de l'exécution dans l'atelier de confections métalliques, du montage sur le chantier ainsi que de l'entretien pendant l'exploitation.

La conception de performance dans la construction, en particulier, dans les constructions métalliques, renferme aussi le critère de la durabilité, c'est-à-dire le maintien de la construction pendant l'exploitation aux caractéristiques initiales de résistance, rigidité et stabilité, établies conformément aux normes techniques. La durabilité des constructions d'acier est influencée sensiblement par son comportement à la corrosion, à la suite de l'interaction entre la construction et le milieu ambiant caractérisé par l'agressivité des agents chimiques.

Au niveau actuel des exigences des constructions il s'agit d'une dépendance entre les mesures de protection contre la corrosion et les performances initiales de la construction.

Les pertes dues à la corrosion, selon certaines statistiques officielles, représentent, approximativement, 5% ... 10% de la production mondiale d'acier. Ceux-ci sont constitués par les dépenses de protection, la réduction de la durée de l'exploitation de la construction, la perturbation ou même l'arrêt de la production pour des travaux de reconstruction.

L'évaluation quantitative de l'effet du milieu chimique agressif sur les éléments structuraux d'acier est nécessaire si l'on tient compte des motivations ci-dessous :

a) Le système de protection anticorrosive (la nature, la composition et le coût) doit être choisi sur la base de la connaissance du degré d'endommagement de la capacité portante de l'élément métallique non protégé par le milieu agressif ; à partir de ce principe on choisit différents systèmes de protection en fonction, par exemple, de la technologie de fabrication de l'acier (produits laminés à chaud, produits formés à froid) ou suivant le type d'assemblage (les vis ordinaires, les vis à haute résistance précontraints, etc.).

b) Les systèmes de protection, adoptés sur la base de l'expérience fournie par la pratique, n'offrent pas une efficacité totale, beaucoup d'accidents (défauts de construction, d'exécution, d'exploitation) peuvent conduire à des dommages locaux du système de protection ainsi qu'à l'accès de l'agent corrosif à la surface de l'élément métallique, la réduction des caractéristiques géométriques, de la résistance et de la stabilité de celle-ci.

c) Dans la littérature de spécialité on recommande des mesures supplémentaires de sécurité en conditions d'exploitation à savoir : choisir une épaisseur minimum de l'élément de la structure, l'augmentation des épaisseurs obtenues par le calcul de résistance, la réduction des résistances de calcul, etc.

d) Dans les travaux de reconstruction de certaines constructions métalliques endommagées par la corrosion il est nécessaire de connaître la capacité portante réelle de celle-ci.

2. La capacité portante des éléments métalliques endommagés par la corrosion

La portion de l'épaisseur de l'élément endommagé par la corrosion ou la profondeur de la pénétration de la corrosion (X — Fig. 1) est déterminée par des mesures directes ou par calcul à partir des relations mathématiques déduites par voie expérimentale.

L'intensité du processus de la corrosion (dX/dt) et l'indice d'endommagement par corrosion (X/t) dépendent du facteur temps (t), des caractéristiques de l'élément d'acier (composition chimique, traitements thermiques, façonnage, assemblages, l'état de surface, etc.), des caractéristiques de l'agent corrosif (nature et agressivité) ainsi que de l'action mécanique (type et

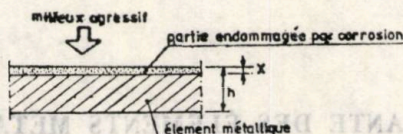


Fig. 1

valeur de la contrainte — σ). Dans la littérature sont indiquées pour la détermination de l'intensité du processus de la corrosion, des relations du type :

$$(1) \quad \frac{dX}{dt} = \varphi(t) \psi(\sigma),$$

où $\varphi(t)$ et $\psi(\sigma)$ sont des fonctions qui dépendent, respectivement, de la variable temps et de la contrainte. Les efforts de tension favorisent la corrosion tandis que celles de compression arrêtent le phénomène.

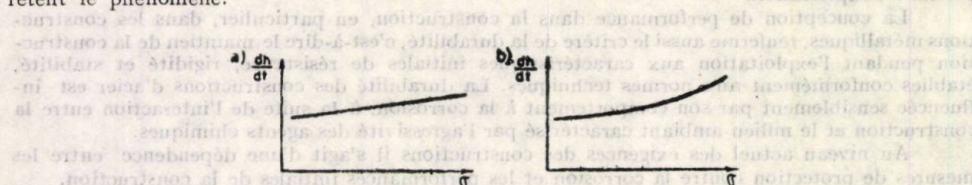


Fig. 2

L'intensité du processus de la corrosion croît avec la tension mécanique. On a montré par voie expérimentale que la contrainte (σ) agit sur le processus de la corrosion à partir de l'instant où sa valeur est supérieure à la limite de fluage de l'acier (σ_c).

Ovtchinnikov [3] a proposé deux modèles mathématiques (Fig. 2) à savoir :

$$(2) \quad \frac{dX}{dt} = \varphi(t)(1 + \beta\sigma), \quad \varphi(t) = \frac{a + b(e^{\alpha t} - 1)}{e^{\alpha t}}$$

et respectivement

$$(3) \quad \frac{dX}{dt} = \varphi(t)\beta\sigma, \quad \varphi(t) = ae^{\alpha t},$$

où a , b , α , β sont des coefficients déterminés par voie expérimentale.

Dans le livre [6], en considérant $\sigma < \sigma_c$, l'auteur a proposé, pour la détermination de la profondeur de pénétration de la corrosion, la relation

$$(4) \quad X = K_1 \ln(K_2 t + 1),$$

où les coefficients K_1 et K_2 , obtenus par voie expérimentale, dépendent des caractéristiques de l'acier et du milieu agressif.

Pour une construction métallique neuve, où il est possible à apparaître le risque de la corrosion et spécialement pour une construction située dans un milieu corrosif (à cause de certains défauts : a — de projet, b — d'exécution, la réalisation mauvaise du système de protection, c — de l'exploitation, si on ne respecte pas les cycles de maintien en bon état ou de la réalisation mauvaise de ceux-ci) il est nécessaire de connaître la capacité portante des éléments structuraux dans chaque instant.

Basés sur les résultats acquis dans un article antérieur [4] on présente ci-dessous une méthode de détermination des caractéristiques géométriques (A , I , W) et de l'effort capable (M_{cap}) d'un élément fléchi en fonction de la profondeur de pénétration de la corrosion (X) et par conséquent de la durée d'exposition à l'agent agressif (t).

Pour une section double T on détermine, par mesure, les dommages absolues (X_{0i}) produites par la corrosion atmosphérique (ces valeurs ne dépendent pas des dimensions de la section, elles dépendent uniquement de la forme de la section, de la position d'exposition dans le milieu ambiant corrosif, de l'agressivité de celui-ci ainsi que de la durée d'exposition) (Fig. 3). En rapportant les valeurs X_{0i} à une valeur de référence, par exemple $X_0 = X_{03} + X_{04}$, on obtient les coefficients α_i . Pour une situation arbitraire, où on connaît $X = X_3 + X_4$ (par mesure ou calcul) on peut calculer les valeurs $X_i = \alpha_i X$ suivant le schéma ci-dessous :

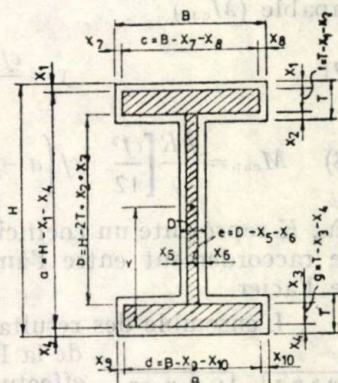


Fig. 3

X_{0i} , [mm/an]; $\alpha_i = \frac{X_{0i}}{X_0}$; $X_i = \alpha_i X$, en obtenant :

$$\begin{aligned}
 (5) \quad & \left\{ \begin{array}{lll} X_{01} = 0,43, & \alpha_1 = 0,755, & X_1 = 0,755X, \\ X_{02} = 0,09, & \alpha_2 = 0,158, & X_2 = 0,158X, \\ X_{03} = 0,45, & \alpha_3 = 0,790, & X_3 = 0,790X, \\ X_{04} = 0,12, & \alpha_4 = 0,210, & X_4 = 0,211X, \\ X_{05} = 0,18, & \alpha_5 = 0,316, & X_5 = 0,316X, \\ X_{06} = 0,18, & \alpha_6 = 0,316, & X_6 = 0,316X, \\ X_{07} = 0,15, & \alpha_7 = 0,263, & X_7 = 0,263X, \\ X_{08} = 0,15, & \alpha_8 = 0,263, & X_8 = 0,263X, \\ X_{09} = 0,15, & \alpha_9 = 0,263, & X_9 = 0,263X, \\ X_{10} = 0,15, & \alpha_{10} = 0,263, & X_{10} = 0,263X. \end{array} \right.
 \end{aligned}$$

À l'aide des valeurs α_i et X on peut exprimer les dimensions de la section endommagée par la corrosion (a , b , ..., q) en fonction des dimensions initiales de la section (B , H , T , D) :

$$\begin{aligned}
 (6) \quad & \left\{ \begin{array}{l} a = H - X_1 - X_4 = H - (\alpha_1 + \alpha_4)X = H - 0,965X, \\ b = H - 2T + X_2 + X_3 = H - 2T + (\alpha_2 + \alpha_3)X = H - 2T + 0,948X, \\ c = B - X_7 - X_8 = B - (\alpha_7 + \alpha_8)X = B - 0,526X, \\ d = B - X_9 - X_{10} = B - (\alpha_9 + \alpha_{10})X = B - 0,526X, \\ e = D - X_5 - X_6 = D - (\alpha_5 + \alpha_6)X = D - 0,632X, \\ f = T - X_1 - X_2 = T - (\alpha_1 + \alpha_2)X = T - 0,913X, \\ g = T - X_3 - X_4 = T - (\alpha_3 + \alpha_4)X = T - X. \end{array} \right.
 \end{aligned}$$

Si l'on connaît les dimensions $a, b, \dots g$ on peut déterminer en ordre : la surface de la section (A), la position du centre de gravité (y), le moment d'inertie (I), le module de résistance au fléchissement (W) et le moment capable (M_{cap}) :

$$(7) \quad y = \frac{cf(2a-f) + be(b+2g) + dg^2}{2(cf + be + dg)}$$

$$(8) \quad M_{cap} = \frac{kR}{y} \left[\frac{cf^3}{12} + cf \left(a - y - \frac{f}{2} \right)^2 + \frac{b^3e}{12} + be \left(\frac{b}{2} + g - y \right)^2 + \frac{dg^3}{12} + dg \left(y - \frac{g}{2} \right)^2 \right],$$

où : K représente un coefficient de correction qui tient compte des portions de raccordement entre l'âme et la semelle ; R — la résistance de calcul de l'acier.

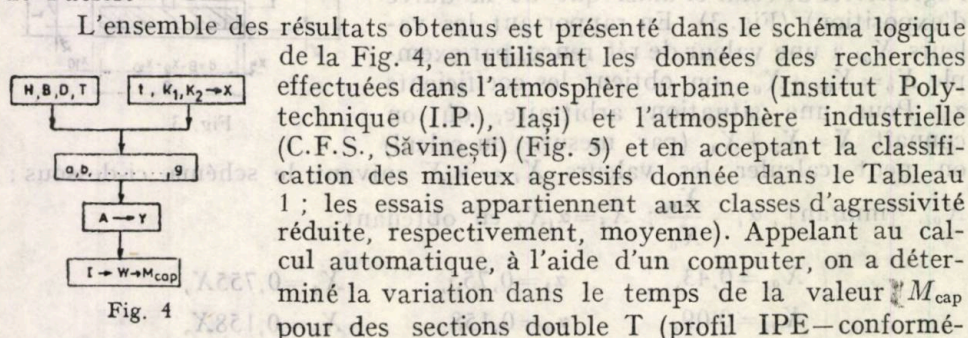


Fig. 4

Tableau 1

L'agressivité du milieu corrosif	L'index d'endommagement par corrosion $\frac{X}{t}$, $\left[\frac{\text{mm}}{\text{année}} \right]$	Observations
Réduite	< 0.1	Pour les essais en atmosphère urbaine (I.P. Iași) — 0,07 mm/année
Moyenne	$0.1 \dots 0.5$	Pour les essais en atmosphère industrielle (C.F.S., Săvinești) — 0,40 mm/année
Puissante	> 0.5	—

Les résultats obtenus permettent d'aboutir aux conclusions ci-dessous :

a) L'agressivité de l'atmosphère d'exploitation de la construction métallique produit un important préjudice par la corrosion et la diminution de la capacité portante ; ainsi, les dommages en atmosphère industrielle (dans le cas analysé constituant la classe d'agressivité moyenne — Tableau 1) sont plus grandes (en pourcentage, six fois) par rapport

Tableau 2

Atmosphère		Urbaine (I.P., Iași)							Industrielle (C.F.S., Săvinești)					
Normes	$t, [annees]$	0	0,5	1	2	5	10	15	0,5	1	2	5	10	15
	X, [mm]	0	0,049	0,070	0,091	0,126	0,151	0,166	0,279	0,401	0,539	0,742	0,898	0,991
Section														
Euro-normes	M_{cap}	718	712	710	707	703	700	699	685	671	654	630	612	601
	daN.m													
	%													
	red. M_{cap}	0	0,8	1,1	1,5	2,1	2,5	2,7	4,6	6,6	8,9	12,2	14,8	16,3
	M_{cap}	4074	4052	4042	4032	4016	4005	3998	3946	3890	3826	3733	3661	3618
	daN.m													
	%													
	red. M_{cap}	0	0,5	0,8	1,0	1,4	1,7	1,9	3,1	4,5	6,1	8,4	10,1	11,2
	M_{cap}	11697	11645	11623	11601	11565	11539	11522	11404	11275	11130	10916	10752	10654
	daN.m													
	%													
	red. M_{cap}	0	0,4	0,6	0,8	1,1	1,4	1,5	2,5	3,6	4,8	6,7	8,1	8,9
Roumaine	M_{cap}	24360	24274	24238	24201	24139	24096	24069	23872	23658	23416	23061	22787	22624
	daN.m													
	%													
	red. M_{cap}	0	0,4	0,5	0,7	0,9	1,1	1,2	2,0	2,9	3,9	5,3	6,5	7,1
	M_{cap}	718	714	712	710	706	704	702	691	678	665	644	628	620
	daN.m													
	%													
	red. M_{cap}	0	0,6	0,9	1,2	1,7	2,0	2,2	3,8	5,6	7,4	10,3	12,5	13,7
	M_{cap}	4494	4476	4468	4460	4446	4437	4431	4389	4342	4290	4213	4154	4118
	daN.m													
	%													
	red. M_{cap}	0	0,4	0,6	0,8	1,1	1,3	1,4	2,3	3,4	4,5	6,2	7,6	8,4
	M_{cap}	13714	13674	13657	13641	13612	13592	13580	13489	13390	13279	13114	12989	12913
	daN.m													
	%													
	red. M_{cap}	0	0,3	0,4	0,5	0,7	0,9	1,0	1,6	2,4	3,2	4,4	5,3	5,8
	M_{cap}	30659	30592	30564	30535	30489	30454	30434	30281	30116	29929	29655	29443	29317
	daN.m													
	%													
	red. M_{cap}	0	0,2	0,3	0,4	0,6	0,7	0,7	1,2	1,8	2,4	3,3	4,0	4,4

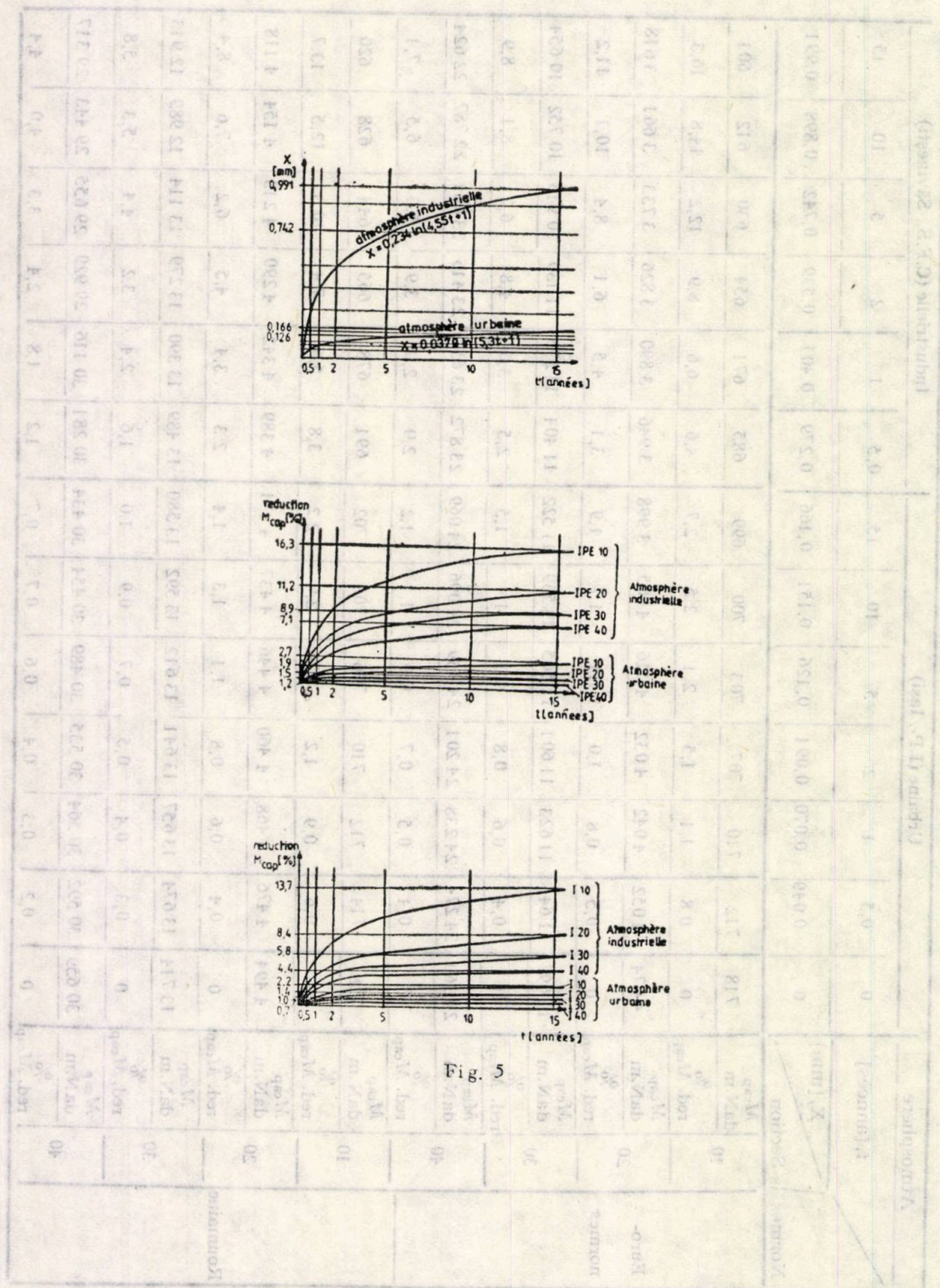


Fig. 5

Table 5

aux celles de l'atmosphère urbaine. Ainsi, on a obtenu : pour le profil IPE (Euronorme) 10 (16,3% en atmosphère industrielle et 2,7% en atmosphère urbaine), IPE 20 (11,2% ; 1,9%) ; IPE 30 (8,9% ; 1,5%) ; IPE 40 (7,1% ; 1,2%) et pour le profil I (produit roumain) 10 (13,7% ; 2,2%), I 20 (8,4% ; 1,4%), I 30 (5,8% ; 1%), I 40 (4,4% ; 0,7%).

b) Le facteur temps produit une corrosion avancée et une diminution sensible de la capacité portante pendant les premiers cinq années d'exploitation et dans une moindre mesure durant la période suivante.

Ainsi, dans les premières cinq années il résulte pour le profil IPE 10 (en atmosphère urbaine 0 ... 2,1% et dans l'atmosphère industrielle 0 ... 12,2%) ; IPE 20 (0 ... 1,4% ; 0 ... 8,4%) ; IPE 30 (0 ... 1,1% ; 0 ... 6,7%) ; IPE 40 (0 ... 0,9% ; 0 ... 5,3%) ; I 10 (0 ... 1,7% ; 0 ... 10,3%) ; I 20 (0 ... 1,1% ; 0 ... 6,2%) ; I 30 (0 ... 0,7% ; 0 ... 4,4%) ; I 40 (0 ... 0,6% ; 0 ... 3,3%). Dans l'intervalle de 5 à 15 années il résulte : pour le profil IPE 10 (2,1 ... 2,7% ; 12,2 ... 16,3%) ; IPE 20 (1,4 ... 1,9% ; 8,4 ... 11,2%) ; IPE 30 (1,1 ... 1,5% ; 6,7 ... 8,9%) ; IPE 40 (0,9 ... 1,2% ; 5,3 ... 7,1%) ; I 10 (1,7 ... 2,2% ; 10,3 ... 13,7%) ; I 20 (1,1 ... 1,4% ; 6,2 ... 8,4%) ; I 30 (0,7 ... 1,0% ; 4,4 ... 5,8%) ; I 40 (0,6 ... 0,7% ; 3,3 ... 4,4%).

c) Les sections ayant des dimensions réduites sont endommagées plus profondément que celle de grandes dimensions. Ainsi, il résulte pour le profil IPE 10 (en atmosphère urbaine après une période d'une année 1,1% et après une période de 15 années 2,7%, en atmosphère industrielle après une période d'une année 6,6% et après une période de 15 années 16,3%) ; IPE 40 [(0,5% ; 1,2%), (2,9% ; 7,1%)] ; I 10 [(0,9% ; 2,2%), (5,6% ; 13,7%)] ; I 40 [(0,3% ; 0,7%) ; (1,8% ; 4,4%)].

d) Le rapport entre les pourcentages d'endommagement pour les profils ayant des hauteurs différentes est maintenu constant dans le temps indépendamment de l'agressivité du milieu corrosif (par exemple, pour le profil IPE ayant les hauteurs de 10, respectivement 40 cm, ce rapport est de 2,2 et pour le profil I ayant les hauteurs de 10, respectivement 40 cm ce rapport est de 3,0).

3. Conclusions

Les éléments constructifs métalliques endommagés par la corrosion par suite des certains dégâts accidentels ou erreurs de projet, d'exécution ou d'exploitation des systèmes de prévention et de protection anticorrosive, ont la capacité portante diminuée, surtout dans les milieux ayant l'agressivité chimique importante.

Il est nécessaire de connaître la modification dans le temps de la capacité portante des éléments de résistance des constructions métalliques due à l'effet de la corrosion.

En partant des résultats obtenus par des essais en atmosphère urbaine (à Jassy) et en atmosphère industrielle (à Săvinești), on propose une méthode de détermination de la capacité portante des éléments métalliques fléchis endommagés par la corrosion, fonction de la durée d'exploitation. L'étude concernant la variation dans le temps du moment capable des sections double T (profil IPE — Euronorme et profil I — normes techniques roumaines) conduit aux conclusions suivantes :

a) L'agressivité du milieu d'exploitation (le cas de l'atmosphère industrielle altérée du point de vue chimique) endommage par corrosion les éléments métalliques et diminue les capacités portantes.

b) Le facteur temps endommage d'une façon sensible, par corrosion, les éléments, dans les premières années de leur exploitation.

c) Les sections de petites dimensions sont beaucoup plus endommagées par corrosion que celles ayant de grandes dimensions.

d) Le rapport entre les pourcentages des dégâts pour les profils ayant des hauteurs différentes, quelque soit l'agressivité du milieu corrosif, reste approximativement constant dans le temps.

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DETERMINAREA CAPACITĂȚII PORTANTE A ELEMENTELOR METALICE ÎNCOVOIAȚE DEGRADATE DE COROZIUNE

(Rezumat)

După evidențierea importanței criteriului de durabilitate în cadrul conceptului de performanță în construcții metalice și sublinierea pierderilor materiale mari cauzate de coroziune, se prezintă o metodă de determinare a capacității portante a elementelor metalice încovoiate, afectate de coroziune (ca urmare a unor degradări accidentale sau a unor erori de proiectare, execuție sau exploatare a sistemelor de prevenire și protecție anticorozivă), în funcție de durata de exploatare în atmosfera urbană sau industrială și deci de adâncimea de pătrundere a coroziunii.

CONTRIBUTIONS À LA DÉTERMINATION DE LA DIFFÉRENCE DES HORIZONS DU THÉODOLYTE DANS LES CYCLES DE MESURE DES DÉFORMATIONS ET DES DÉPLACEMENTS VERTICAUX DES CONSTRUCTIONS PAR NIVELLEMENT TRIGONOMÉTRIQUE DE PRÉCISION

PAR

GH. NISTOR et ANETA BRAIER

1. Introduction

La méthode du nivellement trigonométrique de précision est utilisée pour la mesure de déformations et des déplacements verticaux des constructions étudiées. Dans ce but, en partant des points de station du réseau de référence, on effectue des observations cycliques à l'aide des théodolites de précision, sur les points de contrôle, désignés aussi comme marques à viser, fixées sur la construction. La méthode est appliquée, spécialement, lorsque les points de contrôle des constructions élevées sont éloignés et difficilement accessibles.

Le principe de la méthode de nivellement trigonométrique de précision est basé sur la détermination des altitudes des points de contrôle, qui correspondent à chaque cycle de mesures et le calcul de leur différence qui fournit les valeurs des déformations et des déplacements verticaux. Comme cette procédure suppose un grand volume de calculs, on a recherché des solutions afin de les simplifier et réduire. L'un des auteurs (G.N.) a élaboré une méthode pour déterminer les déformations et les déplacements verticaux des constructions étudiées, fonction directe des différences des angles zénithaux, mesurées cycliquement dans les points des extrémités d'une base fixe, pour le cas général lorsque l'horizon du théodolite change dans chaque cycle d'observations [1], [2].

Dans cette Note on présente une méthode qui permet la détermination directe des différences entre les horizons de l'appareil dans les cycles de mesure, de même que les résultats expérimentaux obtenus, par rapport à la méthode classique.

2. Présentation de la méthode et des résultats expérimentaux

Tant pour déterminer l'horizon du théodolite dans chaque point de station, que pour déterminer la déformation et le déplacement vertical de la construction, la valeur de l'angle zénithal de chaque direction est obtenue à l'aide de 1 ... 3 mesures complètes.

L'horizon du théodolite du point de station A (Fig. 1), qui correspond au cycle initial, est obtenu en utilisant le repère R d'altitude fixe connue, avec la relation

(1)

$$H_A = H_R + S_R - d_A \operatorname{ctg} Z_A.$$

Le repère d'altitude connu établit la surface de référence par rapport à laquelle on mesure les déformations verticales. La lecture S_R sur la mire aura une valeur arrondie, la même dans tous les cycles d'observations. Quand les conditions sur le terrain permettent, la grandeur de cette lecture

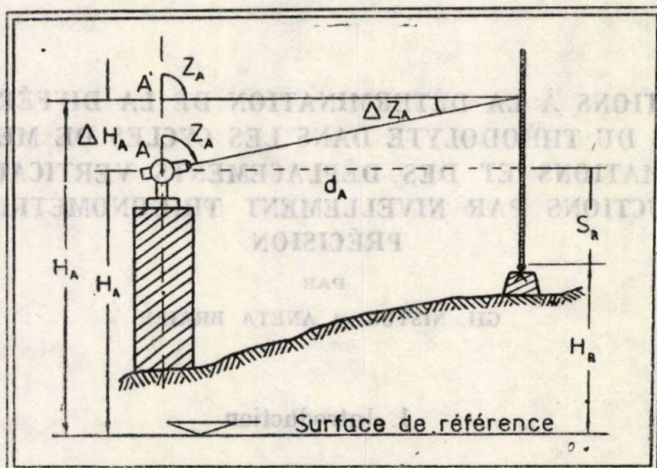


Fig. 1

sera choisie de telle façon que la lunette occupe une position presque horizontale. On aboutit ainsi que l'angle zénithal soit à peu près droit et que l'horizon du théodolite puisse être déterminé à l'aide de la formule simplifiée:

$$(2) \quad H_A = H_R + S_R - \frac{d_A}{\rho^{cc}} (100^g - Z_A)^{cc}.$$

Dans les relations (1) et (2) on a utilisé les notations suivantes: d_A — la distance entre le point de station A et le repère R ; Z_A — l'angle zénithal de la direction.

Quand dans le cycle actuel d'observations on installe de nouveau et on réalise le callage du théodolite, l'horizon de l'instrument se modifie avec une valeur donnée par la relation

$$(3) \quad H'_A = H_R + S_R - d_A \operatorname{ctg} Z'_A,$$

ou

$$(4) \quad H'_A = H_R + S_R - \frac{d_A}{\rho^{cc}} (100^g - Z'_A).$$

La différence entre les valeurs de l'horizon du théodolite dans les deux cycles de mesures, nécessaire pour le calcul des déformations et des déplacements verticaux, sera

$$(5) \quad \Delta H_A = H'_A - H_A = d_A (\operatorname{ctg} Z_A - \operatorname{ctg} Z'_A).$$

Les valeurs des deux horizons étant très approchées, on peut considérer $Z'_A = Z_A + \Delta Z_A$. Si l'on linéarise l'expression de la différence entre les horizons elle devient:

$$(6) \quad \Delta H_A = \frac{d_A}{\rho^{cc} \sin^2 Z_A} \Delta Z_A^{cc}.$$

Quand la lunette est à peu près horizontale, l'angle zénithal étant à peu près droit, on peut écrire la relation (6) sous la forme :

$$(7) \quad \Delta H_A = \frac{d_A}{\rho^{cc}} \Delta Z_A^{cc}.$$

En introduisant dans ces relations les notations :

$$(8) \quad K_A = \frac{d_A}{\rho^{cc} \sin^2 Z_A} \text{ ou } K_A = \frac{d_A}{\rho^{cc}} \text{ quand } Z_A \approx 100^\circ,$$

il en résulte pour la différence entre les horizons la relation finale :

$$(9) \quad \Delta H_A = K_A \Delta Z_A^{cc}.$$

La quantité K_A , constante pour les cycles d'observations, est calculée à l'aide des éléments mesurés dans le cycle initial. La seule quantité variable reste la différence entre les valeurs des angles zénithaux, mesurées dans les cycles d'observations initial et actuel.

La méthode présentée exige que l'angle zénithal de la direction $\overline{AR}$, de même que ceux des directions vers tous les points de contrôle, soient déterminés par trois mesures complètes, dans toutes les deux positions de la lunette, pour tous les trois fils zénithaux : le fil réticulaire horizontal et les deux fils stadimétriques. L'ordre dans la quelle on consigne les résultats de chaque détermination et le calcul de l'angle zénithal moyen est présentée dans le Tableau 1. Dans les Tableaux 2 et 3 on donne les valeurs des mesures effectuées sur le terrain et celles des angles zénithaux moyens, dans les deux cycles d'observations

La différence entre les angles zénithaux moyens, due à la modification de l'hauteur de l'appareil dans les deux cycles d'observations, est

Tableau 1
Détermination de l'angle zénithal moyen

Point de station	Point visé	Cercle gauche (S)	Cercle droit (D)	$2Z = 400^g + S - D$	Angle moyen
A	R	1. Fil bas 2. Fil milieu 3. Fil haut	6. Fil haut 5. Fil milieu 4. Fil bas	$400^g + 1 + 6$ $400^g + 2 - 5$ $400^g + 3 - 4$	$Z = \frac{400 + S - D}{6}$

Tableau 2
Cycle initial

Point de station	Point visé	S_A			D_A			$2Z_A$			Z_A		
		g	c	cc	g	c	cc	g	c	cc	g	c	cc
A	R	99	82	05	299	50	75	200	31	30	100	15	70
		100	13	91	299	82	48	200	31	43			
		100	45	66	300	14	19	200	31	47			

Tableau 3
Cycle actuel

Point de station	Point visé	S'_A g c cc	D'_A g c cc	$2Z'_A$ g c cc	Z'_A g c cc
A	R	99 81 03	299 51 70	200 29 33	100 14 70
		100 12 90	299 83 57	200 29 33	
		100 44 84	300 15 30	200 29 54	

$$\Delta Z_A^{\text{cc}} = Z'_A - Z_A = -100^{\text{cc}}.$$

Pour $d_A = 34\,375$ mm (l'angle zénithal étant mesuré pour une lecture sur la mire de $S_R = 0100$ mm) et $\rho^{\text{cc}} = 636\,620$, on a obtenu pour la différence entre les horizons du théodolite, la valeur

$$\Delta H_A = \frac{d_A}{\rho^{\text{cc}}} \Delta Z_A^{\text{cc}} = -5,4 \text{ mm}.$$

En vue de comparer les résultats on a procédé à la détermination indirecte de la différence des horizons, à l'aide des valeurs moyennes des deux horizons, selon les procédés habituels [3]. Dans chaque horizon on a mesuré les angles zénithaux, avec les deux positions de la lunette, qui correspondent à cinq divisions sur la mire. Dans les Tableaux 4 et 5 sont

Tableau 4
Cycle initial

Station A, Repère R $d_A = 34\,375$ mm, $H_R = 5\,000,0$ mm, $d_A/\rho^{\text{cc}} = 0,053996$					
S_R	Z_I g c cc	Z_{II} g c cc	$Z_I + Z_{II}$ g c cc	$(100 - Z)^{\text{cc}}$	H_A
0100	100 13 85	299 82 49	399 96 34	-1 568	5 184,7
0110	100 11 97	299 84 45	399 96 42	-1 376	5 184,3
0120	100 10 26	299 86 33	399 96 59	-1 196	5 184,6
0130	100 08 32	299 88 01	399 96 33	-1 015	5 184,8
0140	100 06 49	299 89 98	399 96 47	- 825	5 184,6
Moyenne : $H_A = 5\,184,6$ mm					

Tableau 5
Cycle actuel

S_R	Z'_I g c cc	Z'_{II} g c cc	$Z'_I + Z'_{II}$ g c cc	$(100 - Z')^{\text{cc}}$	H'_A
0100	100 12 80	299 83 56	399 96 36	-1 642	5 178,9
0110	100 10 92	299 85 44	399 96 36	-1 274	5 178,8
0120	100 09 10	299 87 22	399 96 32	-1 094	5 179,1
0130	100 07 89	299 89 06	399 96 95	- 911	5 179,2
0140	100 05 40	299 91 03	399 96 43	- 718	5 178,8
Moyenne : $H'_A = 5\,179,0$ mm					

présentés les résultats des mesures effectuées et le calcul de l'horizon moyen du théodolite dans les deux cycles.

La différence entre les horizons du théodolite est

$$\Delta H_A = H'_A - H_A = -5,6 \text{ mm.}$$

On a enregistré une différence de 0,2 mm entre la valeur obtenue par voie directe et celle obtenue par voie indirecte, ce que confirme l'efficacité de la méthode proposée.

3. Calcul de la précision

La différence entre les horizons du théodolite dans les deux cycles, donnée par la relation (7), est fonction des éléments mesurés :

$$(10) \quad \Delta H_A = f(d_A, Z_A, \Delta Z_A).$$

Le carré de l'erreur moyenne quadratique de la différence entre les horizons est

$$(11) \quad m_{\Delta H_A}^2 = \left[\frac{\partial(\Delta H_A)}{\partial d_A} \right]^2 m_{d_A}^2 + \left[\frac{\partial(\Delta H_A)}{\partial Z_A} \right]^2 \left(\frac{m_{Z_A}}{\rho} \right)^2 + \left[\frac{\partial(\Delta H_A)}{\partial(\Delta Z_A)} \right]^2 \left(\frac{m_{\Delta Z_A}}{\rho} \right)^2.$$

En tenant compte de l'expression de ΔH_A , et en introduisant ses dérivées partielles en (11), il en résulte

$$(12) \quad m_{\Delta H_A}^2 = \Delta H_A^2 \left(\frac{m_{d_A}}{d_A} \right)^2 + 4\Delta H_A^2 \operatorname{ctg}^2 Z_A \left(\frac{m_{Z_A}}{\rho} \right)^2 + \left(\frac{\Delta H_A}{\Delta Z_A} \right)^2 m_{\Delta Z_A}^2.$$

On peut mesurer la distance entre les points soit par voie directe, soit indirectement, avec une erreur relative de 1 : 5 000 ... 1 : 10 000. Dans le cas étudié, $m_{d_A} = \pm 5 \text{ mm}$.

L'angle zénithal moyen est obtenu par les trois mesures complètes ($n=3$), ce qui signifie que son erreur moyenne quadratique est

$$(13) \quad m_{Z_A} = \pm \sqrt{\frac{[vv]}{n(n-1)}}.$$

Dans les deux cycles, les erreurs moyennes quadratiques des angles zénithaux moyens ont été $m_{Z_A} = \pm 2^{\text{cc}}, 7$ et $m_{Z'_A} = \pm 3^{\text{cc}}, 7$.

L'erreur de la différence entre les angles zénithaux moyens résulte

$$(14) \quad m_{\Delta Z_A} = \pm \sqrt{m_{Z_A}^2 + m_{Z'_A}^2} = \pm 4^{\text{cc}}, 6.$$

Après l'introduction dans (12) de toutes ces valeurs on a obtenu, pour l'erreur moyenne quadratique de la différence entre les horizons du théodolite, la valeur $m_{\Delta H_A} = \pm 0,2 \text{ mm}$.

En tenant compte que la plus grande influence sur cette erreur exerce le dernier terme, qui contient $m_{\Delta Z_A}$, et de même que le terme K_A étant calculé avec les éléments mesurés dans le cycle initial, reste constant pour tous les autres cycles d'observations, on peut recommander de calculer l'erreur moyenne quadratique de la différence entre les horizons, comme fonction directe de la grandeur mesurée ΔZ_A^{cc} :

$$(15) \quad m_{\Delta H_A}^2 = \left[\frac{\partial(\Delta H_A)}{\partial(\Delta Z_A)} \right]^2 \left(\frac{m_{\Delta Z_A}}{\rho} \right)^2$$

d'où résulte la relation simple

$$(16) \quad m_{\Delta H_A}^2 = K_A^2 m_{\Delta Z_A}^2$$

À l'aide de cette relation on a obtenu pour l'erreur de la différence entre les horizons la même valeur, $m_{\Delta H_A} = \pm 0,2$ mm, ce que confirme la justesse de la recommandation avancée.

4. Conclusions

1. La méthode proposée pour déterminer la différence entre les horizons du théodolite dans les deux cycles d'observations, comme fonction directe de la différence entre les valeurs des angles zénithaux mesurés dans les cycles actuel et initial, s'avère une méthode simple, rapide et commode. Elle assure une très bonne précision dans le contexte de la détermination des déformations et des déplacements verticaux des constructions étudiées par nivellement trigonométrique de précision (méthode élaborée par Gh. Nistor).

2. Les relations obtenues permettent tant le calcul complet de la précision des résultats, que, de même, l'évaluation anticipée de la précision avec laquelle on doit effectuer les mesures des angles zénithaux en terrain, afin qu'on obtienne, avec une précision imposée, la différence entre les horizons.

Requ le 9 janvier 1988

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CONTRIBUȚII LA DETERMINAREA DIFERENȚEI DINTRE ORIZONTURILE TEODOLITULUI DIN CICLURILE DE MĂSURARE A DEFORMAȚIILOR ȘI DEPLASĂRILOR VERTICALE ALE CONSTRUCȚIILOR PRIN NIVELMENT TRIGONOMETRIC DE PRECIZIE

(Rezumat)

Se prezintă o metodă de determinare directă a diferenței dintre orizonturile teodolitului din ciclurile de măsurare a deformațiilor și deplasărilor verticale ale construcțiilor studiate, prin nivelment trigonometric de precizie, împreună cu modul de evaluare a preciziei.

Rezultatele experimentale obținute se compară cu cele din metoda clasică, subliniindu-se avantajele metodei propuse.

CONCRETE STRENGTH EVOLUTION AT MODERATE THERMAL TREATMENTS

BY

SERGIU COROBCEANU, ANGELA VASILESCU, ANTONIE TRELEA and RADU VASILESCU

1. Introduction

The most diverse fragmentation of structures and the forming transfer of reinforced and prestressed concrete members from sites to specialised units have been determined, among others, by the relatively small space of concrete hardening, as well as by the unfavourable influence of climatic factors on this process.

In the conditions of an ever more economical use of production areas and means, there appears as compulsory the acceleration of concrete hardening for which the thermal procedures have proved to be the most adequate.

Thermal treatment of concrete has led, however, to a significant increase in the amount of energy included in the prefabricated members, at variance with the objective necessity of saving any form of energy originating in conventional sources.

For this reason the investigations being carried on concentrate on conventional energy sources out of which, to date, solar energy stands out distinctly, for its conversion into thermal energy is easily obtained and, often, with reduced investments.

However, the thermal treatment of concrete by using solar energy is met only as an experiment, due to both objective and subjective reasons. From the former we will mention only the one referring to the thermal regime with a strong aleatory character. At the same time the thermal treatment using such a source is relatively moderate and allows spectacular increases in concrete strength within a limited time interval.

The aim of these paper is to analyse the evolution of concrete strength at such moderate treatments, ranging between $25^{\circ} \dots 55^{\circ}\text{C}$, associating, at the same time, a technology of manufacturing which should lead to the increase of efficiency of this procedure.

2. Influence of Thermal Treatment on Concrete

It is well known that concrete hardening depends strictly on surrounding temperature and moisture. Admitting $+20^{\circ}\text{C}$ as a normal temperature, its increase will induce an intensification in water and cement reactions, with immediate consequences for concrete strength.

Theoretically speaking, with every 10°C the rate of strength increasing should double as compared to concrete hardening in normal conditions. As a matter of fact, the effect is felt only 1.2 ... 1.7 times [1], due to the destructive phenomena which develop when temperature rises and which leave conspicuously their mark on the structure of concrete thus formed.

Thus, at the age of 28 days, there results a lower strength than that obtained after a normal hardening regime, and the durability characteristics are sensibly affected.

Although it has been discussed [2] ... [4] about the negative consequences of thermal treatment on concrete structures, we will only mention those phenomena which can be acted upon in view of diminishing their effects. Thus:

a) Concrete structure is characterized by a markedly oriented porosity, determined by the migration of moisture from the inside of the member to the outside. The phenomenon will find expression when the temperature of concrete exceeds that of the source, respectively, when the exothermal process of cement, liable to produce draught, is enhanced, i.e. a difference in temperature ranging between 3° and 12°C, depending on the type of cement used and its proportion in the mixture. Within the current thermal treatment technologies, this aspect becomes obvious in the middle of isothermal period, markedly going on throughout the cooling period.

b) The appreciable differentiated expansion of concrete components among which water, the occluded air and gases intervene with the highest thermal expansion coefficients, as the fresh concrete body is gradually losing its free displacement characteristic, determine the occurrence of stresses.

The cement rock, undergoing a full structural process, does not have, yet, sufficiently strong bonds to take over these stresses. The direct consequence is the initiation of a grid of micro- and macrocracks which reduce the strengths of thermally treated concretes.

c) The conclusions of some recent research entirely confirm the results of some well known investigations [2], [3], concerning the density of hydration products which coat as films the cement nuclei. Thus, at temperature rising, the density of these films increases and prevents water from penetrating towards the above mentioned nuclei. These exert, therefore, a screening effect on an important amount by cement grains which practically remain unhydrated, wasting an incredibly large percentage of the chemical binding potential of the agent (25... 60%, depending on the isothermal treatment temperature and the grinding fineness of cement).

d) The non-uniform manner of temperature spreading within the concrete mass characterizes to the full the irretrievably classical technologies which still govern the thermal treatment process. Temperature variations between different concrete member layers constitute another source of new structural defects for well known reasons.

In the light of the above mentioned, it is not surprising for anybody that thermally treated concrete strength is 25... 40% lower than that of concretes with the same composition, but normally hardened. Permeability, freezing and corrosion tests [5] have shown that these features characterizing concrete durability, undergo, most of the time, more important drawbacks than strength.

3. Greenhouse Effect on Concrete

Temperature rise by greenhouse effect compels recognition, of which fossil fuel economy stands out. The development on sites, with quite simple means, of enclosures which would contain prefabricated members of the most varied types and destinations, and where these would be thermally treated by means of solar radiation, may be currently achieved, with minimum investments [7].

However spectacular partial or total fossil energy reduction may seem, in order to be reasonably used, the procedure should become technical-economical even in the conditions of an aleatory and usually moderate thermal regime.

By „technical“ we mean, especially, an acceptable hardening rate and minimum strength losses at the age of 28 days.

In view of reducing the qualitative discrepancy between the thermally treated and the normal concretes, the present technologies recommend a series of measures to be adopted, as: introducing a waiting period, a slow rise of temperature, an increase in cement batching and, sometimes, isothermal treatment at higher temperatures. The provisions are, clearly, uneconomic, and the technical effects do not concord with investments.

We will try to prove, first of all, that the moderately thermal treatment obtained by the greenhouse effect leads to structures with fewer

defects, while using some unimproved concrete compositions. Since temperature rising is sufficiently slow (Figs. 1 and 2) the waiting period in forms is implicitly eliminated.

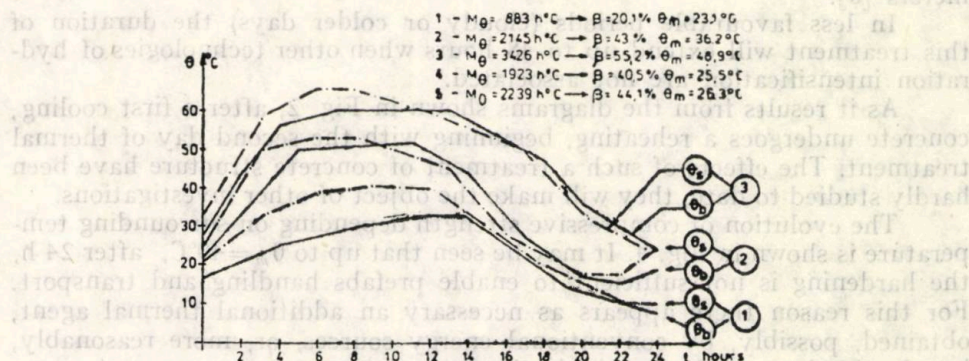


Fig. 1. — Concrete temperature and source evolution during a 24 hours cycle.

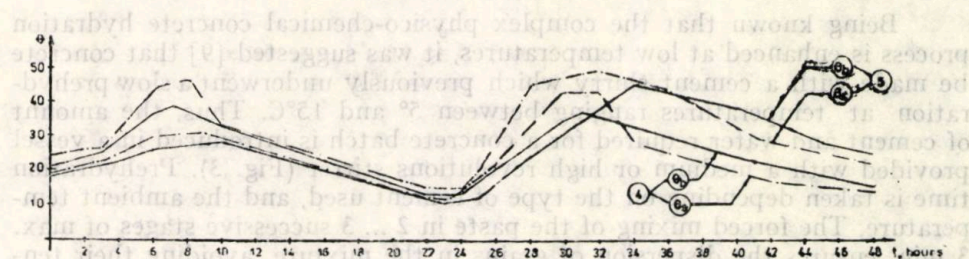


Fig. 2. — Concrete temperature and source variation during a 48 hours cycle.

Concrete experiences, to a lesser degree, the „thermal shock“ and the risks of hydration compounds occurrence do not increase unreasonably, therefore the amount of cement added to the mixture will be more judiciously used. At the same time, due to the moderate temperature at which concrete hardening takes place, the structural defects generated by the differentiated expansion of components are considerably reduced. Finally, the sometimes massive dehydration of concrete, characterizing the common procedure, is prevented by the polyethylene sheet (Fig. 3), covering the cast elements. Even if on its underside moisture condenses when the vapour pressure in the element exceeds that of environment (when $Q_b > Q_s$), in the latter stage of the cycle this situation is reversed and the concrete reabsorbs from environment a great part of the last moisture. Favourable conditions will be thus ensured for cement hydration, while concrete shrinkage will be visibly reduced.

The experiments, conducted on special plots [7], have shown that during March-October period, in sunny days, a 24 hours cycle may prove sufficient to obtain the required strength, which enables handling and transport of the material.

Hardening checking was made at different times, both by the destructive method and by following the maturation process, thus, being once more proved the significant correlation existing between these two parameters [8].

In less favourable periods (cloudy or colder days) the duration of this treatment will extend up to 48 hours when other technologies of hydration intensification are not associated.

As it results from the diagrams shown in Fig. 2, after a first cooling, concrete undergoes a reheating, beginning with the second day of thermal treatment. The effects of such a treatment of concrete structure have been hardly studied to date, they will make the object of other investigations.

The evolution of compressive strength depending on surrounding temperature is shown in Fig. 4. It may be seen that up to $\theta_m = 40^\circ\text{C}$, after 24 h, the hardening is not sufficient to enable prefabs handling and transport. For this reason there appears as necessary an additional thermal agent, obtained, possibly, by conventional energy sources, or, more reasonably, by the implementation of hydration rate increase.

4. Solution for Optimizing the Procedure

Being known that the complex physico-chemical concrete hydration process is enhanced at low temperatures, it was suggested [9] that concrete be made with a cement slurry which previously underwent a slow prehydration at temperatures ranging between 5° and 15°C . Thus, the amount of cement and water required for a concrete batch is introduced in a vessel provided with a medium or high revolutions stirrer (Fig. 3). Prehydration time is taken depending on the type of cement used, and the ambient temperature. The forced mixing of the paste in 2 ... 3 successive stages of max. 3 min. ensures the dispersion of grains in the mixture, avoiding their tendency to agglomerate and transfers hydration from the diffusion layer towards the interior of grains.

After a preestablished time elapse, the cement slurry is poured over the batched aggregates and the concrete mixing and placing follow their normal course. It goes on with thermal treatment at moderate temperatures which enables strength to evolve while, with the classical technologies, it was achieved in much stricter conditions. At the same time, strength losses at the age of 28 days are sensibly reduced.

There was a time when the process was highly appreciated by many investigators, while being confined within its theoretical framework. As batching of materials is done automatically, in concrete mixing plants, the solution complicates somehow the classical batching and mixing process.

Today this relatively simple possibility of intensifying hydration should be, obviously, reconsidered. Cement has become an energo-intensive material and its use has to be made with a maximum effectiveness.

Even for the less finely ground cements, the amount of binder blocked by the screening effect should be as small as possible. On the other hand, prefabrication still involves energy consumptions required for accelerating hardening. Associating cement prehydration to thermal treatment, by the greenhouse effect, may ensure important reductions in conventional energy, with real advantages as to concrete structure and strength.

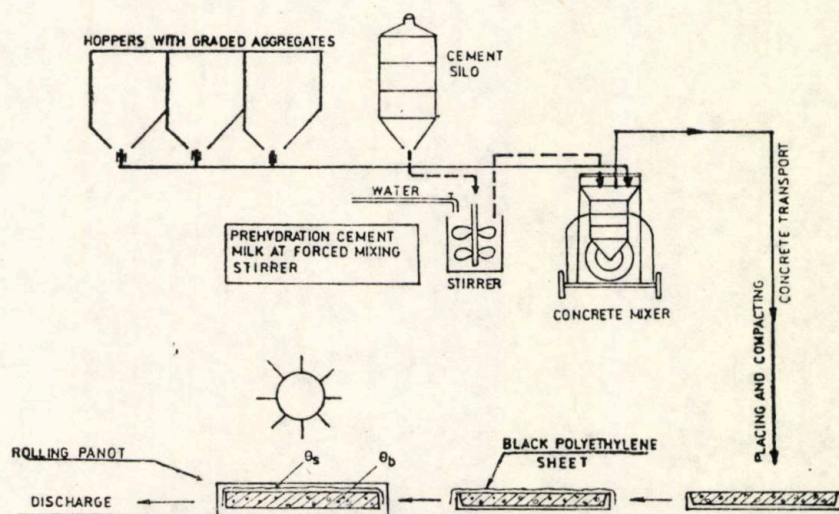


Fig. 3. — Technological layout of concrete preparation and heliothermal curing.

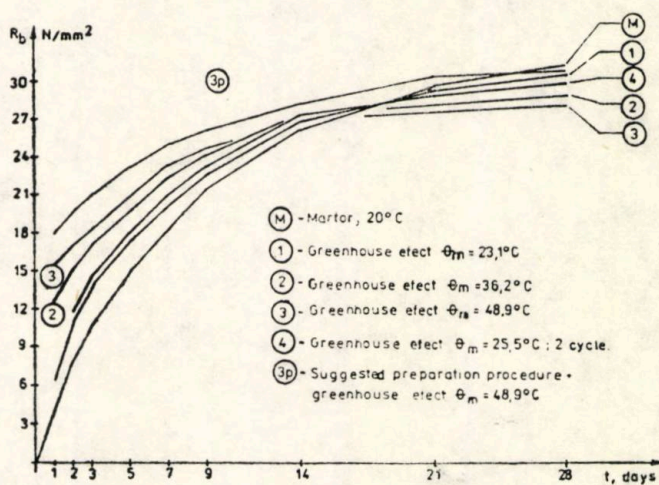


Fig. 4. — Concrete strength evolution depending on preparation technology and surrounding temperature.

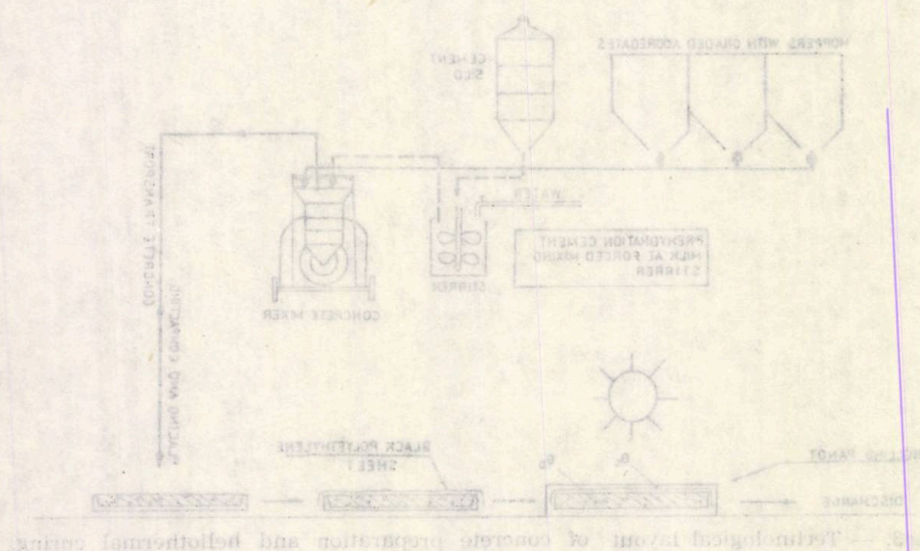


Fig. 3. — Technological layout of concrete preparation and curing.

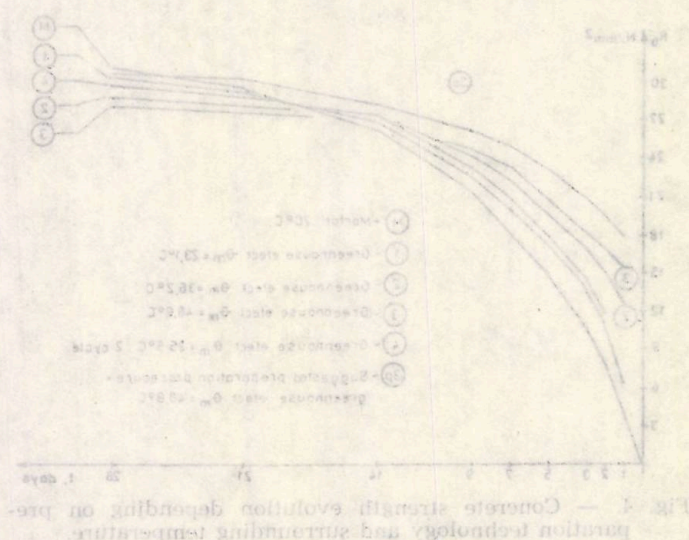


Fig. 4. — Concrete strength evolution depending on curing technology and surrounding temperature.

5. Obtained Results

Out of the series of experiments where the concrete manufacturing process was associated to thermal treatment, only those results were chosen which had been obtained under thermal treatments similar to those obtained by the greenhouse effect.

The maximum temperature of the source was limited to 60°C. The duration of a manufacturing cycle, placing and thermal treating, was limited to 26 hours, the first two hours being affected to prehydration, mixing, placing and compacting operations.

The number of specimens from a batch were thus chosen as to enable, three compressive tests to be conducted at the following dates: 1, 2, 3, 5, 7, 14 and 28 days.

In parallel, strength evolution in the same concrete stored in laboratory conditions was followed simultaneously with that in common concrete batches, identically treated as to temperature.

The graphs shown in Fig. 4 evidence the obtained results [7] in the concrete, where cement Pa 35 was used. It can be seen that only when the average temperature of the source exceeds 40°C (curve 3, $\theta_m = 48.9^\circ\text{C}$) a 24 hours cycle is sufficient to obtain a hardening degree of over 50% (confirmed by the maturation degree as well: $M_\theta = 3\,426\text{ h}^\circ\text{C}$, $\beta = 55.2\%$, the hardening determined by the destructive method $(R_\beta/R_{28})100 = 55.4\%$).

The other thermal regimes do not allow to effect handling-transport operations.

In the mentioned graph there was shown thermally treated concrete strength evolution at $\theta_m = 48.9^\circ\text{C}$, in which cement hydration was activated by the suggested procedure (3p).

During the first five days the strengths were 11% higher as related to those of the commonly made concrete and the strength losses at the age of 28 days, as compared to the witness specimen, were insignificant.

The results obtained for the specimens 1p, 2p and 4p could not find room in the graph, but all prove the efficiency of the procedure.

6. Conclusions

1. Today's energy crisis effects, obviously, the industry of constructions, fully justifying the research conducted in view of reducing energy consumptions in this field of activity.

2. Prefabricated concrete includes an amount of energy which is not negligible from the point of view of binder use and of thermal interventions ensuring its hardening acceleration.

3. Effectiveness of industrialization by prefabrication is indestructibly connected with reducing hardening time of concrete, material which allows the manufacture of most varied members in constructions.

4. From the analysis of today's systems governing this process it was ascertained that there are some reserves as to the way in which binding energy of concrete is used and the possibility of replacing at least in part the conventional energy used for rising concrete temperature.

5. Greenhouse effect solves unexpectedly well this last desideratum in sunny periods with relatively high temperatures, ensuring, after 24 hours, the strengths required for handling the prefabricated elements.

6. For periods of reduced sun light, and not only then, the association of a manufacturing technology is suggested, envisaging cement prehydration, which intensifies the binder-water reactions, with favourable consequences for strength evolution and concrete structure.

7. The suggested procedure limits the thermal treatment duration and leads to an optimal use of the mixed cement, resulting in an increase of strength by 10 ... 22%, as against the common manufacturing procedure.

8. The consumption of energy required for the forced mixing of cement slurry is ascertained to be of max. 15% of the saved energy, what proves, once more, the reason of using this procedure.

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EVOLUȚIA REZISTENȚEI BETONULUI LA TRATAMENTE TERMICE MODERATE

(Rezumat)

Se analizează, pe baza unor date experimentale, efectul tratamentului termic la temperaturi moderate asupra structurii și evoluției rezistenței betonului. Pentru a spori eficacitatea acestui mod de accelerare a întăririi betonului în condițiile utilizării unor surse de energii neconvenționale, se pledează pentru asocierea unei tehnologii de preparare a betonului prin prehidratarea laptelui de ciment.

METHODOLOGY FOR DETERMINING OPTIMUM DIAMETER OF REINFORCEMENTS AND OF THEIR POSITION WITHIN REINFORCED CONCRETE BEAMS

BY

N. FLOREA, V. MUSTĂȚĂ and GH. FLOREA

1. General Considerations

Under tensile stresses the plain concrete, an artificial hard and brittle conglomerate, with mechanical properties similar to natural stone, behaves inadequately. This is manifest, especially by its reduced strengths and ultimate strains. In exceeding them, concrete develops cracks, being no longer able to sustain loads.

In order to make up for the above mentioned drawbacks, it was necessary to associate concrete with reinforcements, especially under the form of steel bars. The assembly concrete—steel, known as reinforced concrete, is a composite material, having higher characteristics than plain concrete. It is made up following the principle that compressive stresses be taken by the concrete and tensile stress by the steel reinforcement.

Along a member, the reinforcements are placed in such a way as to follow the direction of the unit tensile stresses, produced by the action of different types and combinations of loads. Mastering the reinforcing technology assumes a good knowledge of concrete stress distribution and of the influence that reinforcing variants may exert on the strength and cracking capacity of the members.

Beams are structural members subject to bending, characterized by the existence of a tensioned and a compression area separated by a neutral area (Fig. 1). The reinforcing of the tensioned area, within a certain section, implies the solving of two problems: the selection of the optimum bar diameter and the achievement of a certain distribution of these bars on the section so that the reinforcement gravity centre be placed as close as possible to the extreme stressed fibre, thus being observed a series of constructive conditions imposed by standards.

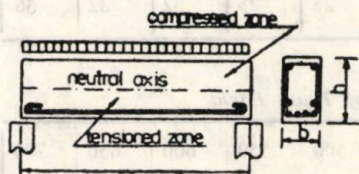


Fig. 1. — Reinforced concrete beam supported at extremities.

2. Choice of Bar Diameter

Although a long experience has been accumulated within decades of calculations and reinforced concrete structure making, up to now there has not been created a methodology allowing to establish the optimum value of bar diameters, depending from the technical and economic criteria, conditioning the structure assembling.

Among these building regulations, only the minimum and maximum permissible values are provided for thus, the longitudinal structure bars for beams are chosen from the standardized assortment (ranging from 10 to 70 mm) without any restriction as to the geometric and mechanical characteristics of their section.

This operation with obvious implications on structural member making is essentially based on the experience and intuition of the constructor.

The automation of reinforced concrete member dimensioning imposes to renounce to the method applied so far and the use of an algorithm able to include as many criteria as possible in the conditioning of the optimum diameter.

Since the relation between bar diameter and some of the cross section geometrical and mechanical characteristics cannot be expressed by mathematical relations, it has been considered that the most indicated way to evidence the influence of these parameters on bar diameter value may be found by statistically interpretation the results obtained as a result of a vast designing experience.

The results of the analysis undertaken may be synthesized as follows :

a) The section height exerts an essential influence on the reinforcement diameter — the diameter values increase as section height increases (Table 1), there existing, however, a limit (a maximum diameter which cannot be exceeded) for each height value (Table 2).

Table 1

Current Diameter Function of Beam Height

H, [mm]	200	250	300	350	400	450	500	550	600
DIN, [mm]	12	12	14	14	16	16	18	20	20

H, [mm]	650	700	750	800	900	1 000	1 100	1 200	1 300	1 400	1 500
DIN, [mm]	20	22	22	25	25	25	28	28	32	32	36

Table 2

Maximum Diameter Function of Beam Height

H, [mm]	200	250	300	350	400	450	500	550	600	650	700
D MAX, [mm]	16	16	18	18	20	20	22	25	25	28	28

H, [mm]	750	800	900	1 000	1 100	1 200	1 300	1 400	1 500
D MAX, [mm]	28	32	36	36	36	40	40	40	40

b) The choice of bar diameter depends on the ratio of section height to breadth, on the reinforcing percentage, on the type of steel and on the quality of concrete. Their influence may be evidenced by means of some coefficients, the importance of which, within the process of finding the optimum diameter, has been estimated after a probabilistic interpretation of a large amount of data. This resulted from an investigation conducted among specialists with a vast designing experience, as well as from the survey of existent documentation. By noting: $CM\emptyset$ — the H/B ratio influence coefficient; $CM1$ — reinforcing percentage influence coefficient; $CM2$ — steel type influence coefficient; $CM3$ — concrete quality influence coefficient, it may be admitted that the global influence exerted by these parameters may be estimated with help of the product of the coefficients affected.

c) The methodology recommended for the finding of longitudinal reinforcement optimum diameter ($D\emptyset L$) consists in taking, in an early stage, an initial value (DIN), function of section height, and multiplying it by a series of influence coefficients:

$$D\emptyset L = DIN \times CM\emptyset \times CM1 \times CM2 \times CM3$$

Table 3

H/B Ratio Influence Coefficients

H/B	1.25	1.50	2.0	2.5	3.0	3.5
$CM\emptyset$	0.8	0.9	1.0	1.05	1.1	1.2

Table 4

Reinforcing Percentage Influence Coefficients

$AP, [\%]$	0.5	0.8	1.0	1.2	1.5	2.0
$CM1$	0.8	0.9	1.0	1.1	1.2	1.3

Table 5

Steel Type Coefficients

Steel type	OB 37	PC 52	PC 60
$CM2$	0.9	1.0	1.20

Table 6

Concrete Quality Influence Coefficients

	$B_c 10$	$B_c 15$	$B_c 20$	$B_c 25$	$B_c 30$	$B_c 35$
$CM3$	1.1	1.06	1.0	0.98	0.97	0.95

In Tables 3 ... 6, for intermediate values, a linear interpolation is being made.

When a dimensioning program is drawn up, it has to be taken into account that some of the design parameters (H , H/B , AP) are not known in the stage of bar diameter measuring. It is, therefore, necessary to resort to an iterative calculation, imposing in the beginning usual values for the unknown magnitudes and checking along calculation the concordance between these and the resulted ones.

Since bar diameter can represent but a standardized value, enclosed in the assortment offered by industry, the algorithm includes the probability that, in case of some fractionary or intermediate magnitudes, to automatically choose, from the vectors of diameters considered, the adequate value.

The process of selecting the bar diameter in a reinforced concrete beam has to meet the economic requirement of covering the necessary reinforcement area resulted from calculation, with as little material in excess as possible (as a rule the choice of diameter and number of bars is made with a surplus of steel).

In order to undertake an analysis, from this point of view, the suggested methodology foresees the possibility to define a number of 3 ... 5 diameters in the vicinity of the optimum one (function of $D\phi L$ position within standardized diameters vector, these are found on one side or the other of $D\phi L$, only to the left or only to the right of $D\phi L$). For each diameter considered, the area of a bar is calculated, as well as the number of bars covering the total reinforcement area and the difference between this and the one corresponding to a whole number of bars. The minimum difference value leads to finding the most economic diameter.

3. Determining Position of Reinforcement Gravity Centre

Within structural and crack growth analysis of reinforced concrete beams, it is of utmost importance to know the position of the gravity centre. The point of application of the unit stress resultant in the tensioned reinforcement which coincides with its gravity centre conditions, the magnitude of the inside couple (Z), the behaviour of the section at exterior loads.

The position of the reinforcement gravity centre as related to the extreme stressed fibre is determined using a static moment equation depending on: total using of a static moment equation number of bars composing the reinforcement (NTB), section breadth (B), bar diameter (D), as well as on a series of prescriptions of making which enable the practical realization of the members (distance between bars (DB), thickness of concrete covering layer (AB)).

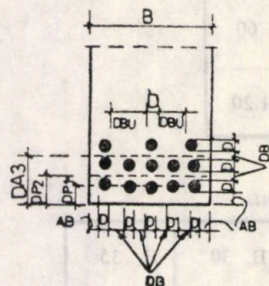


Fig. 2. — Disposition of bars on beam width.

On a rectangular bar cross section having the breadth (B), bars can be disposed on three rows, on the last row the distance between bars (DBU)

being twice as much as the admissible distance on the first two (DB) (Fig.2). This constraint is imposed by the condition of an easier penetration of concrete between bars. Depending on the number of rows on which bars are disposed the position of reinforcement gravity centre as related to adjacent extreme fibre is found using the relations:

a) if bars are disposed on a row :

$$DP1 = AB + 0,5 D ;$$

b) if bars are disposed on two rows :

$$DP2 = [NBR \times (AB + 0,5 D) + NBR2 \times (AB + DB + 1,5 D)] / NTB ;$$

c) if bars are disposed on three rows :

$$DP3 = [NBR \times (DB + 2(AB + D) + NBR3 \times (AB + 2DB + 2,5 D)] / NTB,$$

where: NBR represent the maximum admissible number of bars on the first two rows ; NBU — the maximum admissible number of bars for the last row ; NTB — total number of bars ; NBR2 — incomplete number of bars extant to the second row ; NBR3 — incomplete number of bars for the third row.

The algorithm found determines in an early stage, for a certain amount of reinforcement, the total number of bars (NTB) having the diameter chosen on the basis of before mentioned methodology.

Depending on the beam breadth, it will be studied if these bars may be disposed on one, two or three rows, using the corresponding calculations for each case.

If the reinforcement resulting from the earlier calculation is too strong and exceeds the disposing possibilities offered by a certain section with geometrical characteristics, the decrease of bar diameter will be introduced in the programe. If this provision does not solve the problem, the gradual breadth increase will be resorted to until NBMAX, where :

$$NBMAX = 2NBR + NBU.$$

Similarly, there has been found a calculation procedure for bar gravity centre position, in the case when the bar is splayed on a relatively small area.

Under this situation the distance between the inside forces increases, thus leading to a more rational use of reinforcement and a reduced steel consumption (Fig. 3).

4. Conclusions

The two methodologies presented herein represent contributions to the computerization of reinforced concrete beam design, being part of an ampler activity aiming at complete automation of dimensioning and making these structural members.

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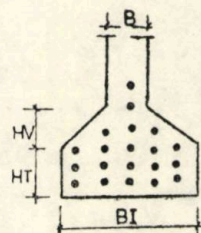


Fig. 3. — Disposition of bars in the splayed basis of a reinforced concrete beam.

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METODOLOGIE DE DETERMINARE A DIAMETRULUI OPTIM AL ARMĂTURILOR ȘI DE POZIȚIONARE A ACESTORA ÎN GRINZILE DE BETON ARMAT

(Rezumat)

Se prezintă o metodologie originală de stabilire a diametrului optim pentru barele longitudinale de rezistență din grinziile de beton armat și de realizare a distribuției acestora pe secțiunea transversală, astfel încât centrul de greutate al armăturii să se găsească cât mai aproape de fibra extremă întinsă.

Conceput algoritmic, pentru a fi transpus cu ușurință în program de calcul, procedeul elaborat ia în considerare la stabilirea diametrului optim al barelor, în afara unor condiții constructive de alcătuire impuse prin norme și o serie de parametri care condiționează nemijlocit mărimea diametrului armăturilor, cum sînt: înălțimea secțiunii transversale (H), raportul dintre înălțimea și lățimea secțiunii (H/B), procentul de armare (AP), tipul oțelului și calitatea betonului utilizat.

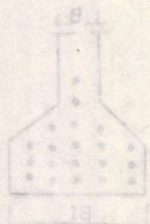


Fig. 1. — Distribuția armăturii în secțiunea transversală a grinzii de beton armat.

The algorithm found determines in an early stage, for a certain amount of reinforcement, the total number of bars (N_{TB}) having the diameter chosen on the basis of before mentioned methodology. Depending on the beam breadth, it will be studied if these bars may be disposed on one, two or three rows. If the reinforcement results from the earlier calculation is too strong and exceeds the disposing possibilities offered by a certain section with geometrical characteristics, the decrease of bar diameter will be introduced in the programme. If this provision does not solve the problem, the gradual breadth increase will be resorted to until N_{BMAX} , where

$$N_{BMAX} = 2N_{BR} - N_{TB}$$

Simultaneously, there has been found a calculation procedure for bar gravity centre position, in the case when the bar is splayed on a relatively small area. Under this situation the distance between the inside forces increases, thus leading to a more rational use of reinforcement and a reduced steel consumption (Fig. 3).

4. Conclusions

The two methodologies presented herein represent contributions to the computerization of reinforced concrete beam design, being part of an amplifier activity aiming at complete automation of dimensioning and mark for these structural members.

ALGORITHM FOR OPTIMAL DESIGN OF PRESTRESSED GIRDERS UNDER STATIC AND MOVING LOADS

BY

N. FLOREA

1. Generalities

The paper presents an algorithm methodology used in the design of prestressed concrete girders having active, linear, adherent reinforcement.

The element is considered to be freely supported at its ends, having an unsymmetrical double T section, constant throughout its length, and acted upon by static gravity loads, as well as by moving live loads, due to bridge cranes.

Assuming that the web width and geometric plate dimensions are known, there may be estimated, on technical-economic grounds the optimum concrete cross section height, as well as the amount of pretensioned reinforcement and its distribution.

At an early stage the optimum height is estimated by calculating the normal sections at bending moments, in the following stages the number is checked at ultimate strength, cracking and strain, both in transfer stages and in service, according to the existing design norms.

The algorithm is conceived in such a way as to enable the study of the influence exerted by the variation of height and concrete quality on the economic feasibility of this make-up.

Since the prestressed concrete member designing process has a marked iterative character, a strict algorithm modulating was resorted to this effect, the calculation blocks repeatedly selected were included in nine subroutines which are called within the main program, as required.

2. Optimization Method Principle

An original methodology of estimating the most economic solutions for the make-up of prestressed concrete girder is presented within this paper by means of up-to-date calculation means and methods.

In solving bent prestressed concrete elements optimizing problem, one assumes that in the maximum moment section, the most economic height (h_e) leads to a minimum cost price function.

For the unsymmetrical double T plate girder (Fig. 1) the cost on unit length is expressed by relation :

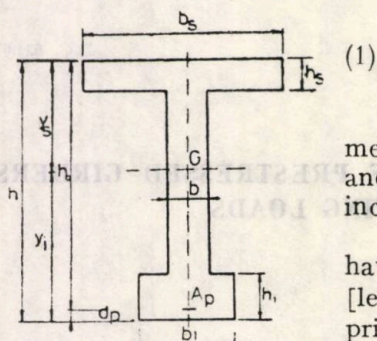


Fig. 1

$$F = [b(h_0 + d_p) + (b_s - b)h_s + (b_i - b)h_i]C_b + \gamma_a A_p C_{ap} + \frac{P_a}{l}, \quad (1)$$

meaning that all expenses concerning labour and equipment, as well as formwork cost are included within material costs.

In relation (1) the following notations have been made: C_b — concrete unit cost price, [lei/m³], C_{ap} — active reinforcement unit cost price, [lei/kg], γ_a — steel specific gravity, [daN/m³], P_a — passive reinforcement total cost price, l — member length, [m].

The study carried out herein aims at optimizing concrete sections and active reinforcement amounts. The passive reinforcement, structurally located, is taken into account on the basis of current consumption recorded in similar elements.

As for schematical section of Fig. 1, if one knows: rib width (h), plate dimensions (b_s , h_s , b_i , h_i) as well as the unpretensioned reinforcement amount and cost, it is seen that the objective function (F) depends on the variation of h_0 and A_p only, i.e.:

$$(2) \quad F = f_1(h_0, A_p).$$

The mechanical equilibrium equations written in normal sections at ultimate strength, which in the present case, are looked upon as restrictions placed on this optimizing problem, allow to make evident the relation between h_0 and A_p . Thus, from the force projection equation one infers:

$$(3) \quad A_p = f_2(x),$$

while from the moment equilibrium equation one finds that:

$$(4) \quad x = f_3(h_0).$$

It results that function $F(h_0, A_p)$ may be expressed under the form:

$$(5) \quad F = f_4(h_0).$$

The active height value leading to a cost function minimum is obtained from equation $dF(h_0)/dh_0 = 0$, which is equivalent to $A + B dX(h_0)/dh_0 = 0$ and represents the optimizing problem solution.

3. Algorithm Description

The high number of parameters intervening in the prestressed concrete design and the limited mathematic apparatus at hand lends to the calculation process a marked iterative character. Since certain operation

blocks are repeatedly run through, it was considered useful to modulate the algorithm, using nine subroutines.

Within the CALAPLA subroutine, the active plate height (b_p) is estimated for the unsymmetrical double T section, transforming, first of all, the real profile into a theoretical one with constant height plates (Fig. 1).

The CALEFUM subroutine is used in measuring the bending moment and the shear force produced by the moving loads in a sufficient number of sections, so that the load envelope graph might be made.

By means of the CALINØP subroutine, the economic active height (h_{oe}) and the corresponding active reinforcement amount are calculated. As initially neither the neutral axis position, nor the working stress in the active reinforcement are known at section failure ($\sigma_{p1} = m_p R_p$) it is necessary to resort to an iterative calculation procedure. Since there have been established, calculation relations of the sections for the cases when the neutral axis crosses the plate and the rib, considering either of the two possible cases, and taking an usual value for the labour coefficient (m_p), one can estimate economic active height, absolute prestressed area height (X_e) and pretensioned reinforcement amount (A_p).

If X_e value, resulting from calculation, defines a neutral axis position in concordance with the assumption made, the dimensioning relations prove to have been accurately used, allowing to calculate the precise value of m_p coefficient.

If its value lies within the vicinity of the initially imposed one, magnitudes h_{oe} and A_p are correctly determined and, consequently, the calculation acquires a final character.

When the resulted m_p coefficient differs from the initially adopted one, obtained after the first iteration, m_p value will be imposed as initial element and the calculations will be remade.

The process will go on until the difference between two successive values ranges between certain prescribed limits.

If the neutral axis was not accurately located by the initial assumption, it is necessary to use dimensioning relations corresponding to its accurate position, thus being possible to ensure, by a similar procedure to the above mentioned one, the agreement between the m_p design and resulting values.

Within the SAPØCEM procedure the exact position of the reinforcement centroid is established for a certain distribution of strands.

At an earlier stage the total number of strands corresponding to the active reinforcement area (A_p) is calculated, continuing with reinforcement distribution in the lower plate, assuming, according to technical norms, the covering concrete layer thickness and the distance between strands. Depending on these magnitudes and the lower plate dimensions, one can to perform the calculus and namely: the distance between strands situated on the same row, the number of strands within a row, the distance between strand rows, as well as the number of strands existing in the plate.

When the reinforcement cannot be located entirely in the lower plate, due to the great number of strands, it is necessary to investigate the conditions under which the strands will be situated on a rib part, as well.

As both the distance between rows and the number of strands existing on each row are known, the distance from the low section fibre p to the reinforcement centroid (d_p) will be, finally, measured, using a static moment equation in relation to the lower fibre.

Initially, the reinforcement centroid position (d_r) was considered known assumption according to which the cross section profile and the pretensioned reinforcement amount could be calculated.

By varying the distance between strands within the prescribed limits, different ways of disposing reinforcement are obtained, each being characterized by a certain position of its centroid (d_p).

It is very unlikely that from all possible situations, the considered one should lead to locating the centroid in the initially admitted position. For this reason, the relation between d_r and d_p will be analysed within the main program.

If a certain tolerance ε is admitted, condition $d_r = d_p \pm \varepsilon$ being unsatisfied, the initially adopted value should be revised, remarking those operations comprised in CALINØP and SAPØCEM until an adequate concordance is reached between the compared values. This process convergence may be favourably acted upon taking the value resulted from calculation as initial datum. For this reason, if $d_r \neq d_p \pm \varepsilon$, the variable change $d_r = d_p$ is operated and the CALINØP subroutine is called.

The CALCARG subroutine is used in establishing the geometric and stiffness characteristics of the section, taking as known values: cross section profile, active and passive reinforcement amount, as well as their location. It will be taken into account that in the supported area the beam presents different cross sections from the central area due to the gradual thickening of the rib to the lower plate width value.

In the CALEFTØ subroutine the loads from dead weight are calculated together with the total loads resulting from the summation of the former and those of the bridge crane determined in the CALEFUM procedure.

The stress valuation is achieved by means of two procedures, due to the fact that initially the dead weight of the element is not known; this can be pointed out only after measuring the section height.

The two total stress components will be determined in various procedures due to the iterative character of the dimensioning method.

In the CALEFAB subroutine the stresses in reinforcement and concrete are determined in transfer stages and in service, for the most favourable loading situations, after a preliminary evaluation of stress losses in the pretensioned reinforcement.

If it is necessary to avoid the appearance of tensile stress on the beam top, in the transfer stage or in service (when the beam is not loaded because the bridge is in the neighbouring span) a pretensioned reinforcement amount will be introduced in the prestressed concrete area.

In order to estimate the normal stress within concrete and reinforcement in the second stage, necessary for normal crack opening calculations, in those areas where there is the required pretensioned reinforcement, Hooke's law is admitted throughout the cross section.

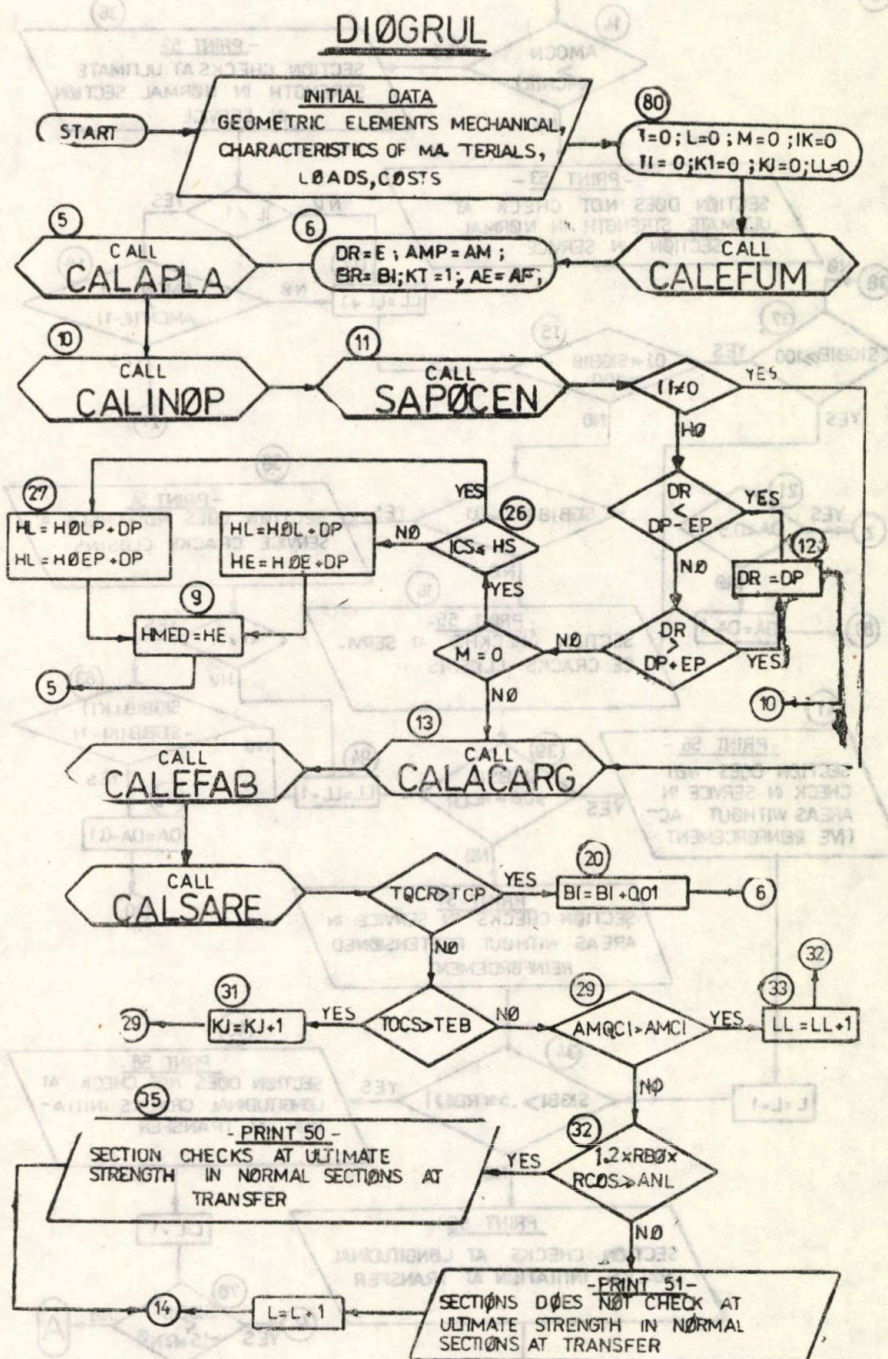


Fig. 2

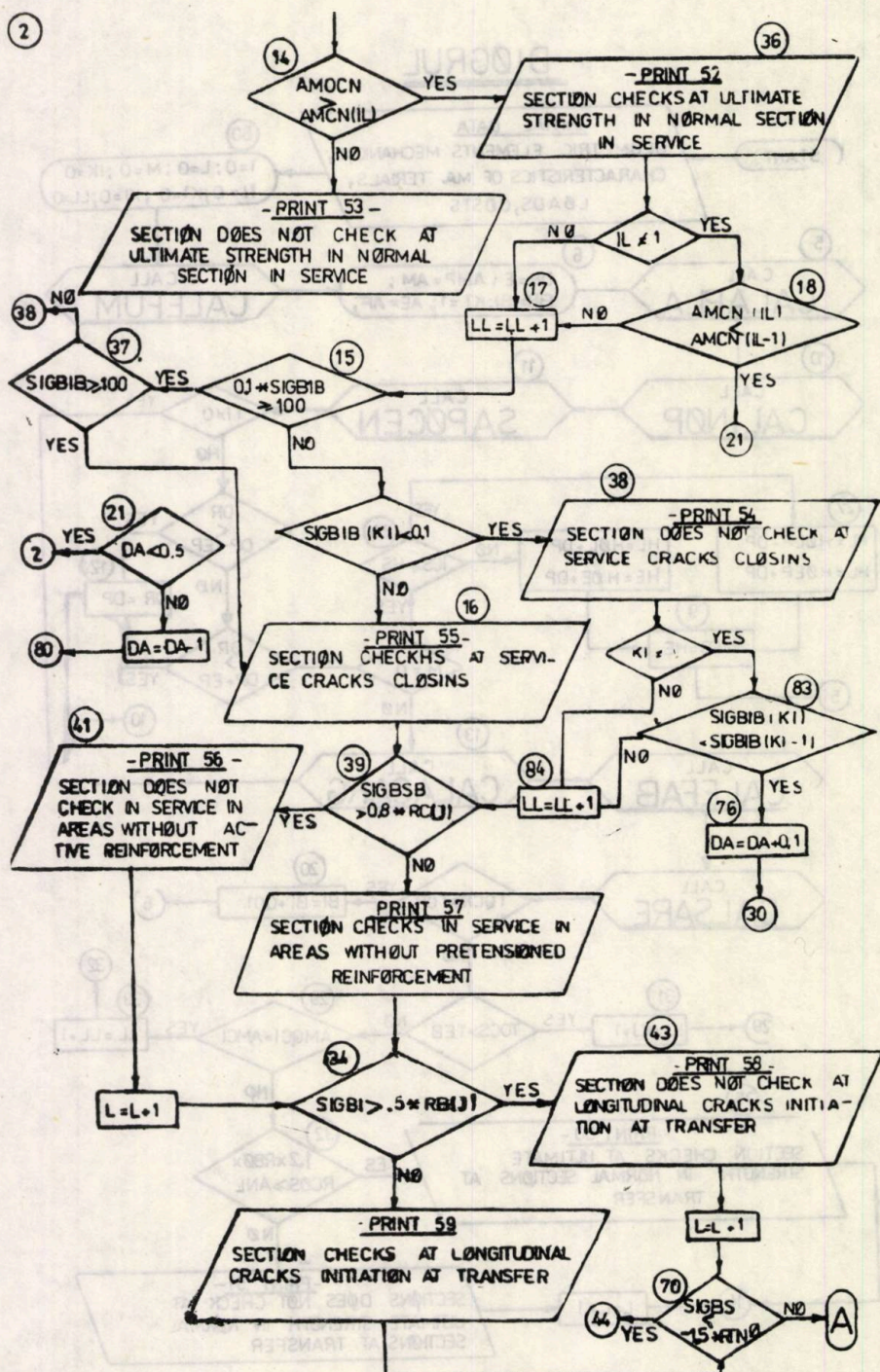


Fig. 3

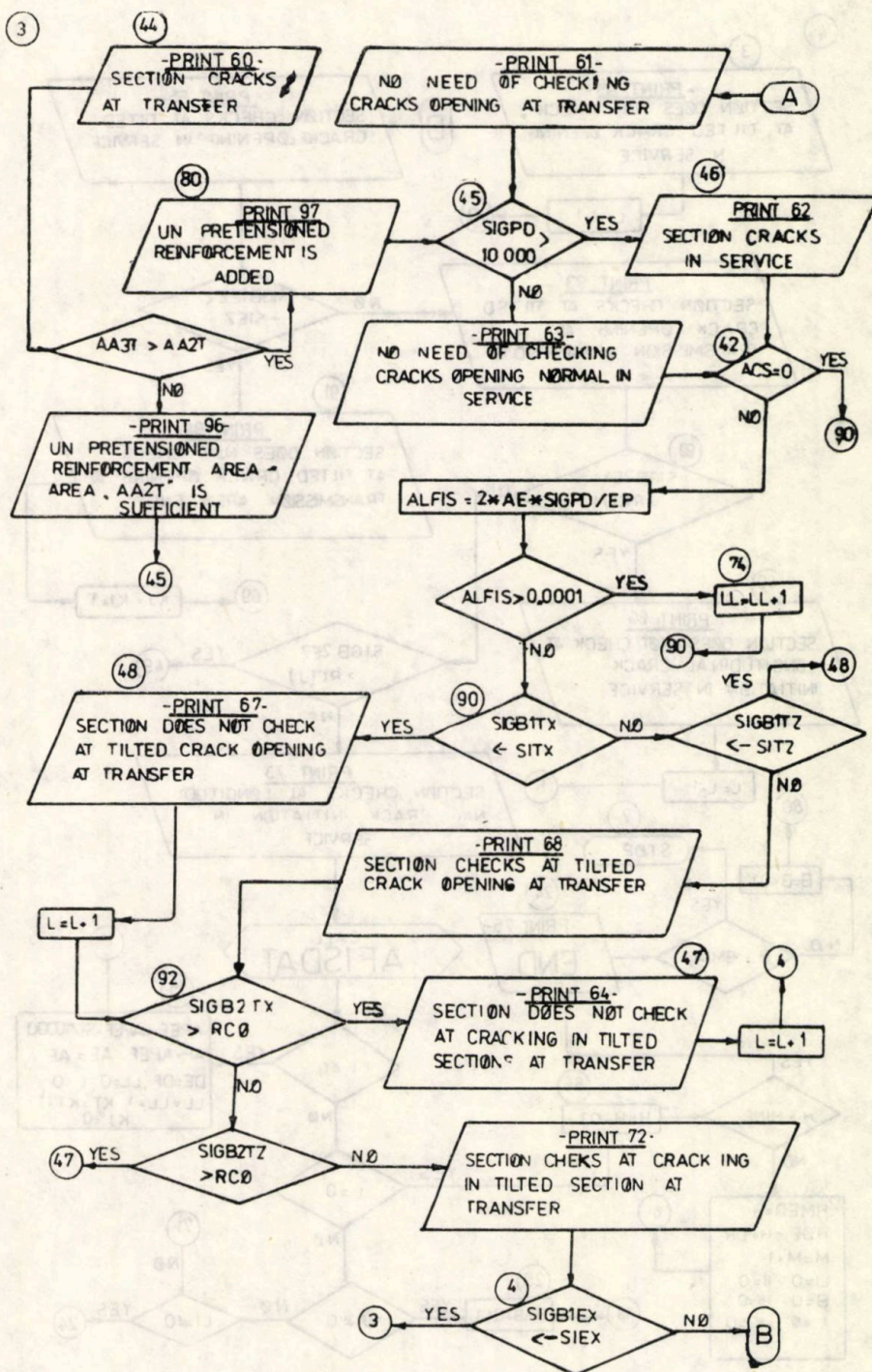


Fig. 4

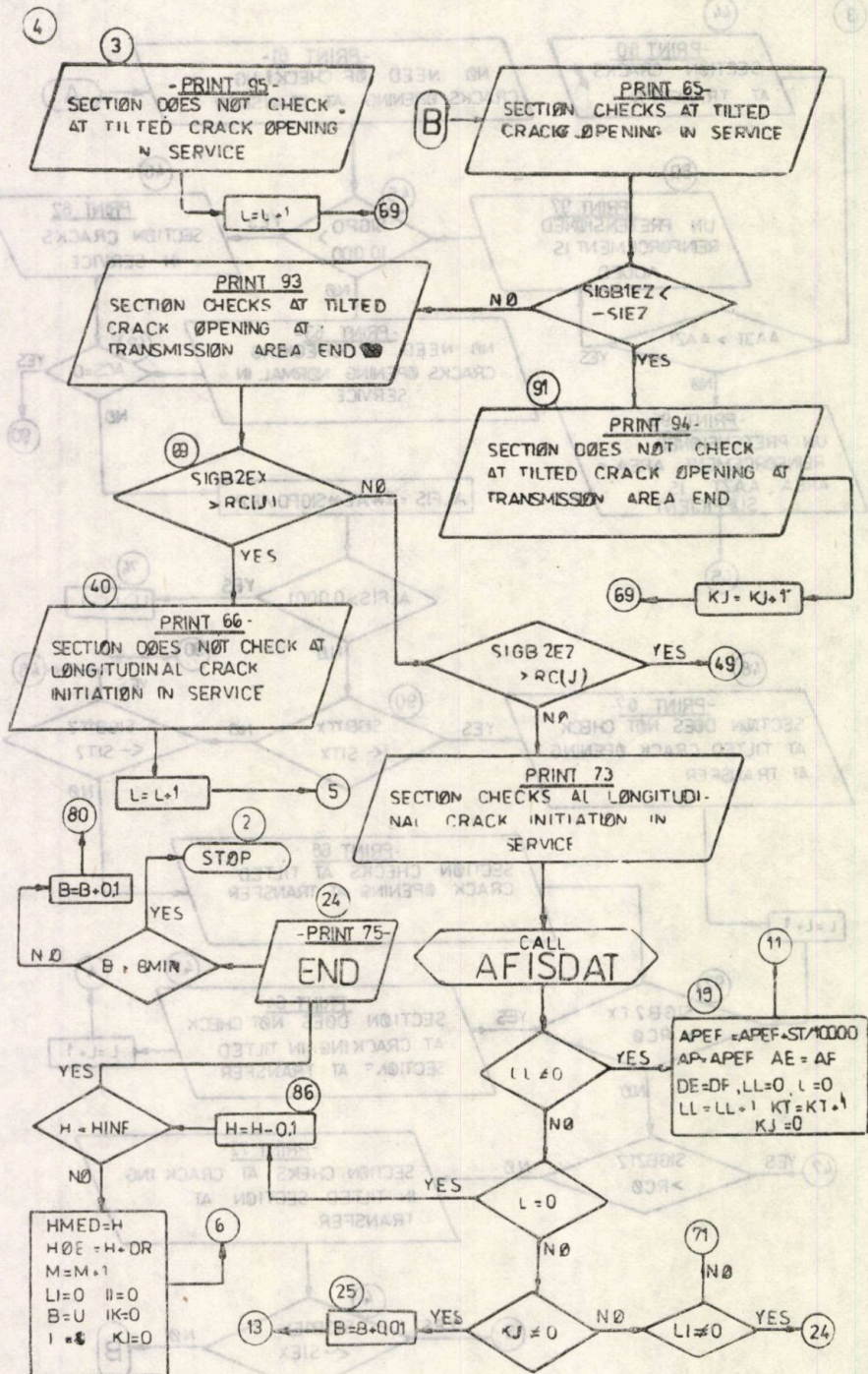


Fig. 5

The solving of the cubic polynomial equation which defines the neutral axis position after crack initiation is achieved by means of RESØLEC subroutine, using *B a i r s t r o w* method.

Within the CALSARE subroutine the required elements are prepared for carrying out the check-up at ultimate strength in normal and tilted sections, at transfer and in service. The normal section calculation in service will take into account the longitudinal brake force of the bridge, thus leading to admitting an excentric tensile stress.

The calculation of tilted sections under shear forces aims at determining the transverse reinforcements as stirrups (diameter, number of branches, distance between stirrups).

On the basis of data provided by the above mentioned procedures, within the main program there are conducted check-up required by norms for ultimate strength, cracking, strain, fatigue and strength. When a certain limit state is not satisfied, it will be conveniently acted upon the geometrical and mechanical characteristics conditioning the section behaviour in the situation considered herein. In this aspect the following measures have been adopted: a gradual increase of pretensioned reinforcement amount; addition of an unpretensioned reinforcement amount to the beam top in those areas close to supports; increase of stirrup diameter and decrease of distance between them, change in concrete quality.

Initial data or calculated values modifying are achieved within the strict limit frame supposed by the technical norms.

The algorithm allows to study the influence exerted by height variation related to optimum value, on economic and technical characteristics of the element.

The AFISDAT procedure as made up of a succession of writing instructions and is used in displaying the initial data as well as the final results.

The proposed algorithm is represented graphically in Figs. 2 ... 5.

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ALGORITMIZAREA PROIECTĂRII OPTIME A GRINZILOR PRECOMPRIMATE ACŢIONATE DE ÎNCĂRCĂRI STATICE ŞI MOBILE

(Rezumat)

Se prezintă un program de calcul care serveşte la determinarea înălţimii optime a secţiunii, a cantităţii de armătură pretensionată şi a modului său de distribuţie în grinzile de rulare din beton precomprimat. Se folosesc în acest sens criterii tehnice şi economice, pe baza cărora este asigurată rezistenţa, stabilitatea şi durabilitatea elementului în condiţiile obţinerii unor cheltuieli de producţie minime.

Sînt prevăzute facilităţi pentru studierea influenţei pe care o exercită variaţia înălţimii şi marca de beton asupra economicităţii soluţiei de alcătuire.

Secţiunea optimă rezultată din calcul se verifică la stările limită de rezistenţă, fisurare şi deformaţie, în faza de transfer şi exploatare, conform prescripţiilor de proiectare existente.

The solving of the cubic polynomial equation which defines the neutral axis position after crack initiation is achieved by means of RESQLEC subroutine, using Bairstow method.

Within the CALSARE subroutine the required elements are prepared for carrying out the check-up at ultimate strength in normal and tilted sections, at transfer and in service. The normal section calculation in service will take into account the longitudinal brake force of the bridge, thus leading to admitting an eccentric tensile stress.

The calculation of tilted sections under shear forces aims at determining the transverse reinforcements as stirrups (diameter, number of branches, distance between stirrups).

On the basis of data provided by the above mentioned procedures, within the main program there are conducted check-up required by norms for ultimate strength, cracking, strain, fatigue and strength. When a certain limit state is not satisfied, it will be conventionally acted upon the geometrical and mechanical characteristics conditioning the section behaviour in the situation considered herein. In this aspect the following measures have been adopted: a gradual increase of prestensioned reinforcement amount; addition of an unpensioned reinforcement amount to the beam top in those areas close to supports; increase of stirrup diameter and decrease of distance between them; change in concrete quality.

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Polytechnic Institute, Iasi,
Department of Structural Mechanics,
Concrete and Technology

Iasi, February 22, 1988

ALGORITMIZAREA PROIECTĂRII OPTIME A GRINTELOR PRECOMPRIMATE ACȚIONATE DE ÎNCĂRCĂRI STATICE ȘI MOBILE

(Rezumat)

Se prezintă un program de calcul care servește la determinarea înălțimii optime a secțiunii, a cantității de armătură prestensionată și a modului său de distribuție în grinzile de beton din beton precomprimate. Se folosesc în acest scop criterii tehnice și economice, pe baza cărora este asigurată rezistența, stabilitatea și durabilitatea elementelor în condițiile optime de proiectare.

Sunt prezentate facilități pentru stabilirea influenței pe care o exercită variația înălțimii și a mărții betonului economisită la soluții de calcul.

Secțiunea optimă rezultată din calcul se verifică la stările limită de rezistență, flambaj și deformație, în țara de transfer și exploatare, conform prescripțiilor de proiectare existente.

NEW RESEARCH ON CONCRETE HARDENING IN RELATION TO ITS MATURATION DEGREE

BY

ANTONIE TRELEA, ANGELA VASILESCU and SERGIU COROBCEANU

1. Introduction

Reinforced or prestressed concrete structural members manufacturing includes a series of processes as formwork removal, precast members handling, prestressing force transfer and, last but not least, the chance of concrete getting frozen during cold weather work. For these reasons, concrete is subjective to mechanical loads liable to unfavourably influence its quality, i.e. mechanical strengths which should be relied on in service and durability.

Since, due to economic reasons, the above mentioned processes take place before concrete has reached the performance degree allowed for in the structural member design, expressed by its compressive strength R_{28} and measured on specimens kept under standard ambient conditions, the technical prescriptions stipulate that these processes should develop only after concrete has reached certain hardening degrees, β , expressed in turn by the percentage of effective strength in concrete at the given moment, R_{β} and R_{28} .

The special importance of the problem lies therefore in that the untimely carrying into effect of the above mentioned processes has negative influences on quality, while their unjustifiable delay brings forth economic prejudice by the wasteful stagnation of production means and areas.

Minimum term evaluation, when the above mentioned processes may take place without impinging on concrete quality, although it is a major feature of this material technology, has not been found yet, due to its complexity, but for a partial solution through recommendations given in literature, of using in this respect the concept named „maturation degree“ [1], [2].

The above mentioned statement is grounded on the fact that in the quoted works neither explanations nor adequate scope is given to the influence of such significant factors as: a) type of cement used, since the relation hardening percentage β vs. maturation degree M_{β} , experimentally found, varies only depending on concrete compressive strength at 28 days, although concretes prepared from different cements, but possessing the same strength R_{28} , have various hardening velocities; b) concrete maturation variability M_{θ} at temperatures θ different from normal ($+20^{\circ}\text{C}$), for which maturation degree M_{β} has been experimentally estimated and its relation to hardening degree β [1], [2].

2. Previous Research

Previous research conducted within the Civil Engineering Faculty, Polytechnic Institute Iași, [3] have emphasized the significant influence of these factors and have led to establishing some correlations β vs. M_{β} for two types of cement, namely: Pa 35 and P 40, commonly used in Romania, as well as to correlating the maturation degree M_{β} at $+20^{\circ}\text{C}$ with the maturation degree M_{θ} , at a temperature $\theta = 5^{\circ} \dots 30^{\circ}\text{C}$, with a relation having the form:

$$(1) \quad M_{\beta} = M_{\theta} K_{\theta} = (\theta + 10) M_{\beta} K_{\theta},$$

where K_{θ} — correlation coefficient between the maturation degrees M_{β} and M_{θ} .

At concrete temperature values varying up to $\pm 10^{\circ}\text{C}$ against the normal one ($+20^{\circ}\text{C}$), the correlation coefficient K_{θ} undergoes variations which do not exceed $\pm 10\%$ as related to $K_{\theta} = 1.0$, which corresponds to a temperature $\theta = +20^{\circ}\text{C}$, allowing the neglect of this factor for the above mentioned temperature interval [1], [2].

The great importance conferred by concrete technology to this problem has made the teaching staff to continue the research program [3].

3. Research Programme

The new research program has had in view :

a) to determine the correlation hardening degree β vs. maturation degree M_β (at normal temperature) for another type of cement with a lower hardening velocity (M 30) ;

b) to check-up the relation between the maturation degree M_β at normal temperature and M_0 at a temperature ranging up to $+50^\circ \dots +60^\circ\text{C}$;

c) to elaborate a method for the estimation of the maturation degree which may allow of taking into account the influence of temperature variation, without letting the method become too laborious.

The experimental programme consisted in manufacturing 264 cubic specimens with 14.1 cm. sides, made of 0/16 mm gravel concrete in three sorts, with 425 Kg/m³ cement M 30 and $W/C=0.48$, grouped, depending on storage conditions and testing age, under the following series :

1° A_0 — specimens kept under standard conditions and tested at the age of 1 ; 2 ; 3 ; 5 ; 7 ; 9 and 28 days, being estimated the R_β and R_{28} parameters.

2° $A_1 \dots A_4$ specimens, thermally treated at average temperatures of $28.6^\circ \dots 36.0^\circ \dots 43.1^\circ$ and 52.9°C respectively, kept relatively constant for 1 ; 2 ; 3 ; 5 ; 7 and 9 days and tested at the end of their thermal treatment, being thus measured the parameter R_β .

3° $D_1 \dots D_4$ specimens treated thermally in the same way, but kept after that under standard climatic conditions until they were tested at 28 days in order to measure the parameter R_{28}^* .

4. Experimental

By means of these experimental results there have been found, for the A_0 series, the following parameters :

$$(2) \quad \beta = \frac{R_\beta}{R_{28}^{20}} \cdot 100 \text{ and } M_\beta = (20 + 10)t_\beta$$

and for the $A_1 \dots A_4$ series, using the R_{28}^* values of the $D_1 \dots D_4$ series, the parameters :

$$(3) \quad \beta^* = \frac{R_\beta}{R_{28}^*} \cdot 100 \text{ and } M_0 = (\theta + 10)t_\beta.$$

At the same time there has been determined a decrease of the final strength coefficient K_r , as a result of thermal treatment at the mentioned temperatures, using the relation :

$$(4) \quad K_r = \frac{R_{28}^*}{R_{28}^{20}}.$$

4.1. β vs. M_β and β vs. M_θ Correlations

The β vs. M_β and β^* vs. M_θ correlations have resulted from the statistical processing of those parameters found using the relations (2) and (3) under the form of some linear regressions between the values β vs. $\log M_\beta$ and β^* vs. $\log M_\theta$ respectively, considering the logarithms of M_β , M_θ as parameters in order to obtain linear equations.

The above mentioned parameters have been grouped within five selections, function of storage conditions and testing ages, establishing the following results:

a) B_0 selections, where there could be grouped the β vs. M_β values from A_0 series, for $\beta \leq 70\%$, the regression equation being:

$$(5) \quad \beta = a + b \log M_\beta.$$

b) $B_1 \dots B_4$ selections, where there could be grouped β^* vs. M_θ values from $A_1 \dots A_4$ series, also for $\beta \leq 70\%$, the regression equation being:

$$(6) \quad \beta^* = a_i + b_i \log M_\theta.$$

Substituting in (5) M_θ , which results from (1), one can obtain:

$$(7) \quad \log K_\theta = \frac{a_i - a}{b} + \frac{b_i - b}{b} \log M_\theta.$$

It may be seen that for $b_i = b$ the coefficient K_θ does not depend on M_θ , but only on a_i , a and b , so that, after determining the regression equations (5) and (6) and checking their linearity (F -test) there could be, also, verified the likelihood of unifying the slope coefficients (the t -test applied for checking-up the significance of slope coefficient differences) the result being positive.

In the case of $B_1 \dots B_4$ selections the significance of selection media difference has been, however, checked (together with T -test), resulting the possibility of recombining B_1 and B_2 selections characterized by average concrete temperatures $\theta_{bm} = 28.6^\circ\text{C}$ and 36°C , respectively, in a single selection, there being registered the temperature $\theta_{bm} = 32.2^\circ\text{C}$.

The coefficients obtained for the regression equation are shown in Table 1, together with average temperatures and K_θ coefficients determined using the relation (7), the regression equation being plotted in Fig. 1.

Table 1

Selections	Regression equation coefficients for				θ_{bm} °C	K_θ
	$x = \log M$		$x = \log 10^{-3} M$			
	a	b	a	b		
Cement M 30 $\beta \leq 70\%$	B_0	-134.3		19.3	20	1.00
	B_{1-2}	-129.6		24.0	32.2	1.23
	B_3	-121.2	51.2	32.4	43.1	1.80
	B_n	-114.5		39.1	52.2	2.43

The β vs. M_β correlation, at normal temperature for M 30 cement, as compared to the same parameters for Pa 35 and P 40 cements, is shown in Table 2, wherefrom significant differences between their hardening velocities may be drawn.

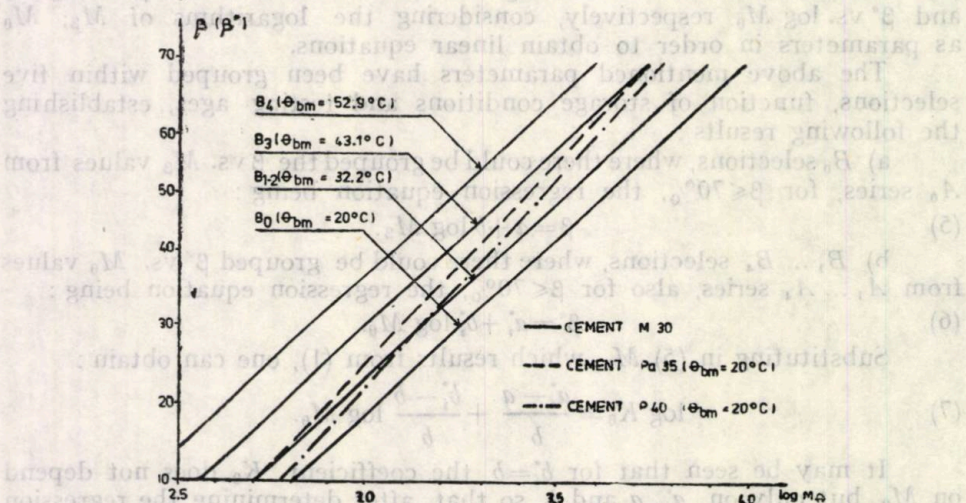


Fig. 1. — Correlation β vs. $\log M_\beta$.

Table 2

Cement type	Maturation degree M_β , [h°C], for hardening degree β , [%]						
	10	20	30	40	50	60	70
M 30	660	1 030	1 620	2 540	3 980	6 240	9 790
Pa 35	600	880	1 290	1 880	2 760	4 050	5 930
P 40	520	740	1 150	1 690	2 510	3 720	5 520

4.2. Correlation K_θ vs. θ

Variation of parameter K_θ vs. θ has been measured by devising, with Table 1 values, a C selection for which the linear regression equation has the form:

$$(8) \quad \log K_\theta = -0.364 + 1.42 \times 10^{-3} \theta.$$

This equation enables to measure K_θ at temperatures ranging from $\theta = 25^\circ\text{C}$ to 60°C , with deviations from experimental values not being in excess of $\pm 1\%$. By virtue of the relatively reduced number of terms in C selection, this accuracy agrees with a probability of $P = 82\%$.

For higher probabilities, the unfavourable permissible deviations may attain $\pm 5.72\%$ ($P = 90\%$) and $\pm 11.74\%$ ($P = 95\%$). Anyhow, the results can be considered satisfactory in view of the objectives aimed at.

4.3. t_β Durations

Estimation of t_β durations for imposed degrees of hardening becomes relatively simple if the correlations β vs. M_β and K_θ vs. θ are known. In this respect, a substitution will be made in relation (1) namely:

$$(9) \quad m_\theta = (\theta + 10)K_\theta,$$

obtaining:

$$t_\beta = \frac{M_\beta}{P_\theta}.$$

The m_θ values will be tabulated into an auxiliary table (Table 3), while the linear interpolation may be used for intermediate values.

Table 3

$\theta (\theta_i)$	1	5	10	15	20	25	30	35	40	45	50	55	60
$m_\theta (m_i)$	3	11	19	25	30	35	46	61	80	104	134	170	212

With thermal regimes in which concrete temperature varies nonlinearly for relatively small intervals, the duration t_β will be discretized into time intervals, t_i , when it can be admitted that temperature is linear. In this situation both average temperatures θ_i and their corresponding values m_i (Table 3) can be calculated.

Duration t_β results from the expression:

$$(11) \quad t_\beta = \sum_i t_i, \quad (11)$$

for which the condition:

$$(12) \quad m_i t_i \geq M_\beta$$

is fulfilled.

4.4. Correlation K_r vs. M_θ

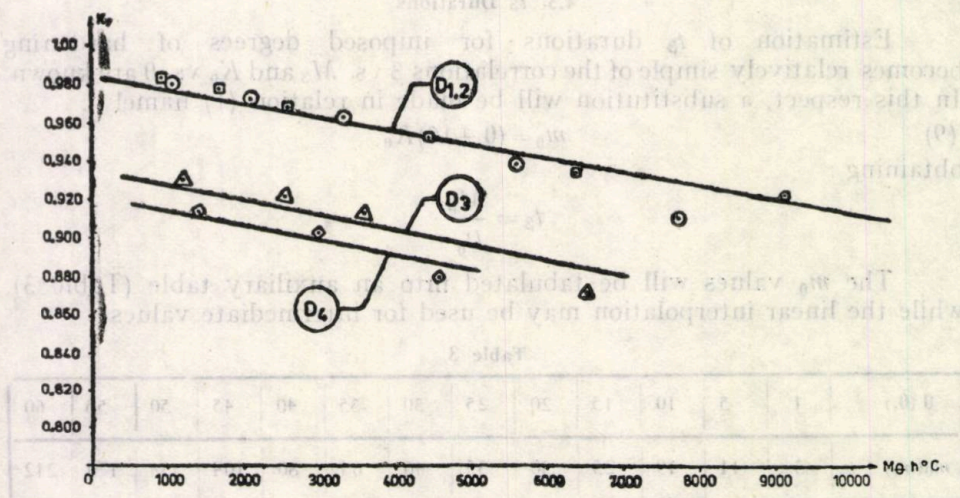
Decrease of concrete final strengths when hardened at higher temperatures than normal constitutes a noteworthy aspect, since it aggravates with temperature and curing time increase, both of these parameters being mirrored in the maturation degree M_θ (Fig. 2).

By determining the correlations β vs. M_β and β^* vs. M_θ (instead of β vs. M_θ as it seemed necessary at first sight) the final strength losses elucidation was avoided, it remaining required only in concrete design. This aspect might be further explained by the following observations:

a) The actual quality of a concrete accounted for in dimensioning or testing a structural element will materialize in the parameter R'_{28} reflecting the influence of hardening conditions, as well (temperature, duration). Concrete hardening should be therefore expressed by β^* (rel. (3)).

b) In these circumstances the designed concrete should enable, under normal hardening conditions, a higher strength R'_{28} , in order to make up for the strength losses due to thermal treating, expressed by the coefficient K_r (4). It results that:

$$(13) \quad R'_{28} = \frac{R_{28}}{K_r}.$$

Fig. 2.—Correlation K_r vs. M_0 .

c) The hardening degree β imposed by technical prescriptions does not refer to that concrete listed at point b), but to a fictive one, whose strength, under normal hardening conditions, will be:

$$(14) \quad R_{28} = R_{28}^*.$$

d) Therefore, it results that the hardening degree of the concrete listed at point a) (β'), will be equal to the that of the concrete listed at point c) (β), both being higher than β' for the concrete listed at point b), i.e.

$$(15) \quad \beta = \beta^* = \beta' K_r,$$

where K_r —final strength decrease coefficient.

5. Conclusions

1. The problem of determining minimum terms t_b when alterations can be allowed of structural elements liable to mechanical loadings, remains a major aspect in concrete technology.

2. Research carried into effect so far enables to ascertain that this aspect can be solved quite simply and effectively by virtue of the maturation degree concept.

3. The integral solution requires, however, the continuation of research for the completion of experimental data and correlations, concerning both the cements used and the variable temperature regime influence.

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NOI CERCETĂRI PRIVIND CORELAREA GRADULUI DE MATURIZARE AL BETONULUI
CU NIVELUL SĂU DE ÎNTĂRIRE

(Rezumat)

În vederea generalizării folosirii conceptului gradului de maturizare al betonului, cu scopul controlul nivelului de întărire, se prezintă cercetările efectuate pe cimentul M 30 care completează studiile anterioare efectuate pe cimenturile Pa 35 și P 40, inclusiv corelarea grade-
lor de maturizare la temperaturi variabile cu cele la temperatură normală.

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NOI CERCEȚĂRI PRIVIND CORELAȚIA GRADULUI DE MATURARE AL BETONULUI CU NIVELUL SĂU DE ÎNĂLȚIRE

(Rezumat)

În vederea generalizării folosirii concepției gradului de maturare al betonului, ca scopul controlului nivelului de înălțire, se prezintă corelațiile electrice pe cimentul M 30 care compunează studiile anterioare efectuate pe cimenturile P 35 și P 40, inclusiv corelațiile grade-
for de maturare la temperaturi variabile cu cele la temperatură normală.

UN NOUVEAU TYPE DE STRUCTURE FILTRANTE EN ARC RÉALISÉE PAR DES ÉLÉMENTS PRÉFABRIQUÉS, POUR LES TRAVAUX TRANSVERSAUX UTILISÉS DANS L'AMÉNAGEMENT DES FORMATIONS TORRENTIELLES

PAR

N. UNGUREANU et N. POPOVICI

1. Introduction

Par suite des multiples avantages que la préfabrication leur offre dans le domaine des constructions, tout récemment on a commencé à réaliser des différents types de structures d'éléments préfabriqués de petites dimensions, pour les ouvrages transversaux, destinés à retenir les alluvions sur les réseaux torrentiels.

Dans ce sens on connaît la structure filtrante en arc, réalisée à l'aide de deux types de préfabriqués prismatiques en béton, avec une forme spécifique qui permet un assemblage efficient sans l'utilisation du mortier ou de l'armature, mais la solution constructive présente la difficulté d'impliquer la nécessité d'utiliser deux types de coffrages pour l'exécution des préfabriqués.

Aussi, on a proposé la réalisation d'une structure avec un seul type de préfabriqué en béton, mais possédant une stabilité relative jusqu'au colmatage des alluvions (la formation de l'atterrissement).

2. Présentation de la structure proposée

On peut réaliser le nouveau type de structure, que l'on propose ci-dessous, à l'aide d'un seul type de préfabriqué (1) (Fig. 1 ... 5) prismatique, en béton, muni avec un orifice-barbacane (c) (Fig. 1) ayant le diamètre de 80 ... 120 mm (dans la zone déversée), pour l'évacuation de l'eau et aussi pour faciliter la manoeuvre [4].

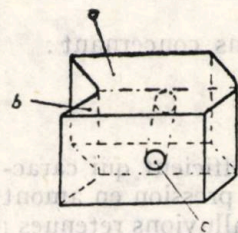


Fig. 1. — L'élément préfabriqué.

Les préfabriqués sont placés sur le fondement (4) (Fig. 2 ... 5) en béton, disposés en rangs horizontaux en forme d'arc. Au-dessus du dernier rang, dans la zone du déversoir et dans les ailes latérales, est réalisée une couche de béton (3) (Fig. 2 ... 4) par coulage.

L'élément préfabriqué (Fig. 1), réalisé sous la forme d'un corps résulté de la liaison par jonction, sur leurs

faces longues de deux prismes (a) et (b) obliques (Fig. 1), ayant comme bases des parallélogrammes, est exécuté en forme d'un seul bloc par coulage et présente les faces verticales qui viennent en contact avec d'autres préfabriqués du rang, l'une sous forme de surface convexe et l'autre de surface concave, de telle manière que pour la structure réalisée, en rapport avec les modes d'arrangement des préfabriqués, la transmission des efforts soit réalisée en deux directions et quelquefois même en trois directions (le fonctionnement en console).

On peut réaliser la structure en plusieurs variantes, selon la manière d'arranger les préfabriqués et l'ouverture de l'emplacement dans le quel on construit l'ouvrage transversal. De la sorte, dans la Fig. 2 l'arrangement des préfabriqués commence dès la clef de l'arc, l'opération de montage en se réalisant d'une façon symétrique. Dans la zone de clef de l'arc le finissage doit être réalisé avec du béton monolithe (5) (Fig. 2).

Dans deux autres variantes de réalisation de la structure (Fig. 3) les éléments préfabriqués sont rangés dans une seule direction dans tous les rangs, le cas où le bloc monolithe est tourné dans la zone de la clef de l'arc en devenant inutile.

Dans la troisième variante, qu'on peut appliquer aux barages des lits des torrents avec une grande largeur, en vue de réduire la flèche des arcs, on construit plusieurs voûtes qui se soutiennent sur des contreforts (6) (Fig. 4); on obtient, dans ce cas, des barrages aux voûtes multiples.

Pour réaliser une bonne interdépendance des éléments préfabriqués dans la structure et pour obtenir une plus grande précision pendant l'exécution, on recommande à couler ces éléments en coffrages qui doivent réaliser l'arc du terrain du barrage. Pour obtenir une liaison qui soit capable à transmettre les efforts en trois directions, dans la Fig. 5 on présente le détail de montage (7) (Fig. 4) des éléments préfabriqués, disposés décalés l'un par rapport à l'autre sur la verticale.

L'encastrement des deux extrémités de la structure aux rivages se réalise à l'aide d'un béton monolithe (2) (Fig. 2...4) évasé au contact avec le terrain, afin que les efforts transmis à celui-ci soient inférieures comparativement aux ceux admissibles.

3. Problèmes de calcul et d'exécution

Les préfabriqués doivent être réalisés en béton, la marque B 200 ... B 300, compacté par vibration. À l'exécution on va assurer le caractère plan des joints, horizontaux et verticaux.

À l'état de projet on va effectuer des vérifications concernant :

a) La compression :

$$(1) \quad p_{\max} r < 2mR\lambda,$$

où 2λ représente la grosseur du préfabriqué; m — le coefficient qui caractérise les conditions de travail ($m=0,8 \dots 0,9$); $p_{\max}$ — la pression en amont à la base de la structure, due à la pression de l'eau et des alluvions retenues; R — la résistance de calcul à la compression du béton; r — le rayon moyen de courbure de la structure.

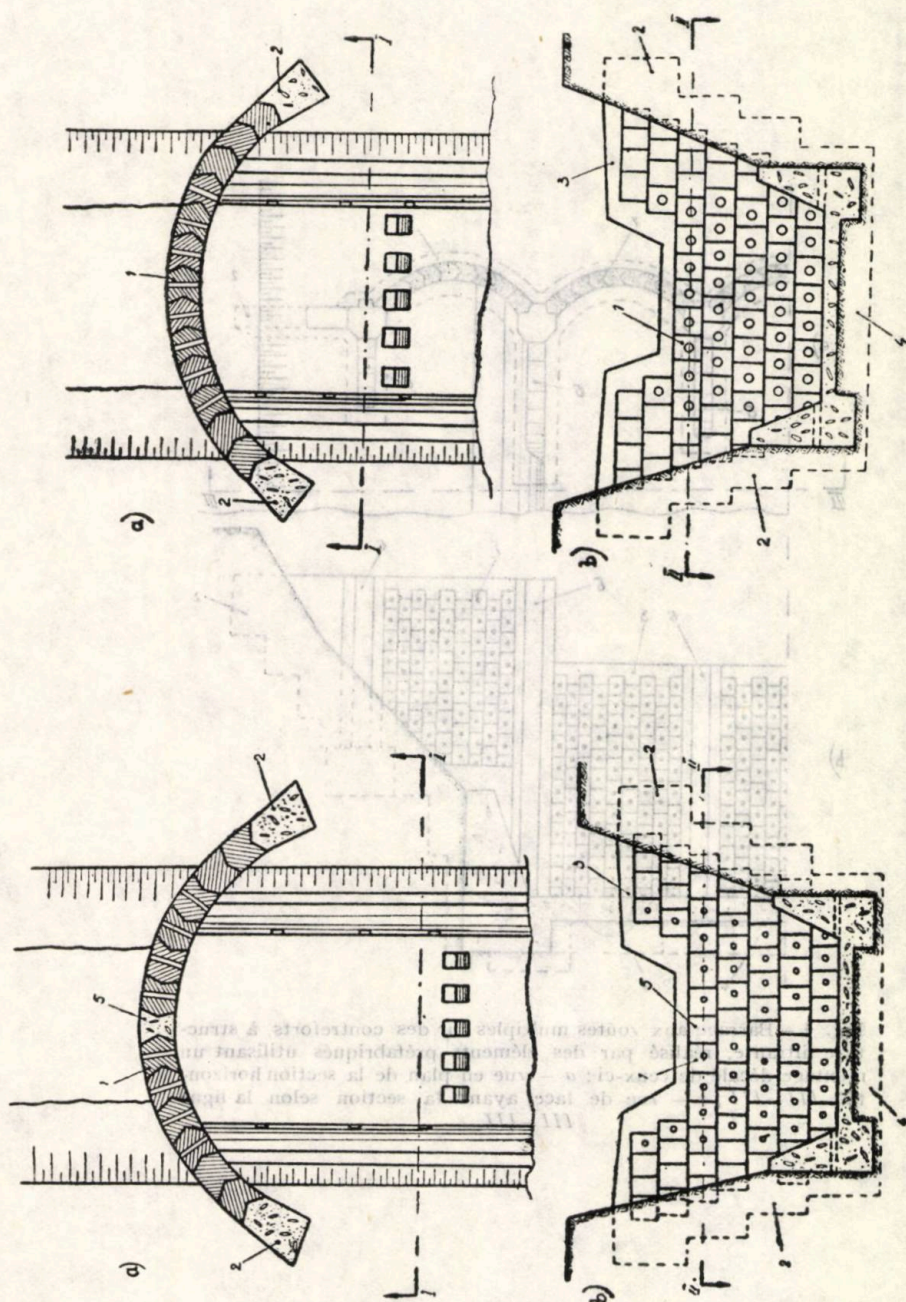


Fig. 2. — Barrage en arc à structure filtrante réalisé par des éléments préfabriqués (variante I) : a — vue en plan de la section horizontale I—I ; b — vue de face ayant la section selon la ligne I—I.

Fig. 3. — Barrage en arc à structure filtrante réalisé par des éléments préfabriqués (variante II) : a — vue en plan de la section horizontale II-II ; b — vue de face ayant la section selon la ligne II-II.

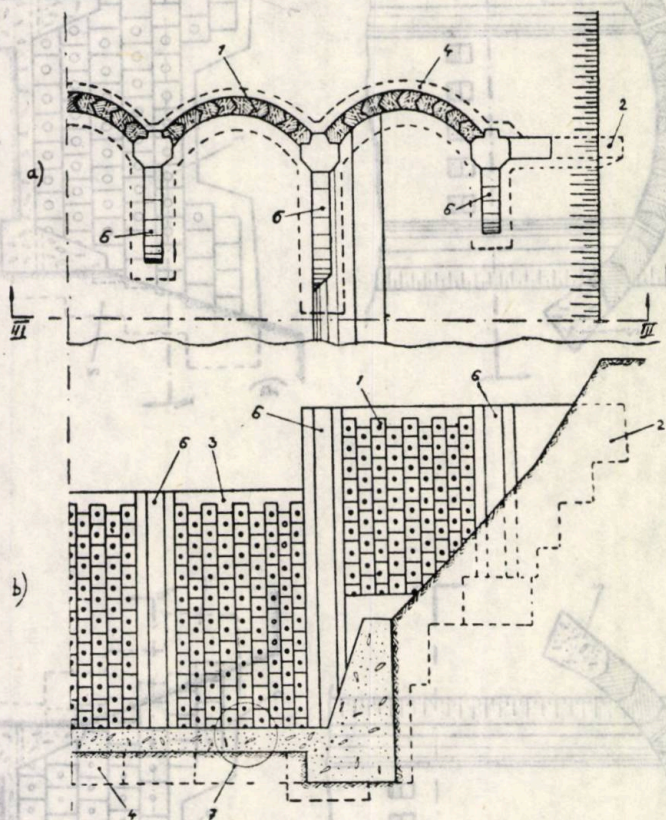


Fig. 4. — Barrage aux voûtes multiples sur des contreforts, à structure filtrante, réalisé par des éléments préfabriqués utilisant un montage décalé de ceux-ci : a — vue en plan de la section horizontale III—III ; b — vue de face ayant la section selon la ligne III—III.

b) Le cisaillement :

$$(2) \quad p_{\max} a' h \leq 2 m_1 \lambda h R_f,$$

où : R_f — la résistance de rupture au cisaillement du béton ; m_1 — un coefficient qui prend en considération la diminution de la section au cisaillement ($m_1 = 0,85 \dots 0,90$).

La dimension λ résulte d'habitude de la condition de cisaillement du préfabriqué le plus sollicité (Fig. 6). Compte tenu que durant l'exploitation peuvent apparaître des diverses situations, on ne va pas adopter $\lambda < 15$ cm, la grosseur de la structure (2λ) étant comprise entre 30 ... 50 cm.

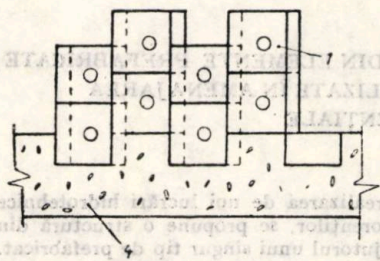


Fig. 5. — Détail de montage décalé des éléments préfabriqués.

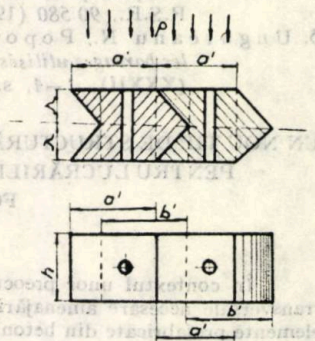


Fig. 6. — Détails des éléments préfabriqués pour la structure proposée.

Prenant en considération le poids relatif petit des préfabriqués (entre 50 ... 110 kg) on peut les exploiter pour des hauteurs de 4 ... 8 m avec des moyens usuels de mécanisation.

D'autres problèmes concernant l'exécution sont similaires aux autres types de structures en arc réalisés par des préfabriqués et ont été mentionnés dans nos ouvrages antérieurs [2] ... [5].

4. Conclusions

L'utilisation de la structure en arc avec des éléments préfabriqués en béton présenté ci-dessus permet à obtenir les avantages suivants :

1. L'augmentation de la qualité et du degré de l'industrialisation à l'exécution des éléments en polygone et aux ouvrages transversaux en terrain.

2. La réduction du temps de travail et en même temps l'évitement de l'utilisation des moyens mécanisés lourds pour l'opération de montage en terrain.

3. La réalisation plus simple de la structure en terrain.

4. La réduction de la consommation de matériaux et du coût du travaille en ensemble.

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UN NOU TIP DE STRUCTURĂ FILTRANTĂ ÎN ARC DIN ELEMENTE PREFABRICATE,
PENTRU LUCRĂRILE TRANSVERSALE UTILIZATE ÎN AMENAJAREA
FORMAȚIUNILOR TORENȚIALE

(Rezumat)

În contextul unor preocupări constante privind realizarea de noi lucrări hidrotehnice transversale necesare amenajării rețelei de scurgere a torenților, se propune o structură din elemente prefabricate din beton, posibil de executat cu ajutorul unui singur tip de prefabricat.

Se expun mai multe variante constructive ale soluției, având indici tehnico-economici avantajoși.

THE PRESSURE STATE ANALYSIS IN EXISTING WATER SUPPLY NETWORK IN VIEW OF SERVICE PRESSURE ASSURANCE

BY

EMANOIL BÂRSAN, CĂLIN IGNAT and CONSTANȚA BÂRSAN

1. Introduction

An existing water supply network for an urban area has a known geometry (plane disposition and diameter dimensions) resulted from the performed design for a maximum hour flow rate and the service pressures established in function of the number of inhabitants, the comfort degree and the adopted building system.

Time passing, there can be added new consumers to the existing network that modify the initial pressure state. As a result it must be done an analysis in view of flow rate augmentation and service pressure assurance.

This paper deals with the pressure state determination in the existing water supply networks in the urban areas with the aim of adopting ways for service pressure assurance.

2. The Used Procedure

Because of the particularity of problem the analysis was performed considering as unknowns the piezometric heads at nodes.

The continuity of discharge at nodes is expressed by the relation:

$$(1) \quad f_j = \sum_{i=1}^N q_{ij} + C_j = 0, \quad (j=1, \dots, N), \quad (N - \text{number of nodes}),$$

where: f_j represents the algebraic sum of discharges in node j (unbalance) q_{ij} — pipe line discharge ij with the sign $+$ for discharges that enter the node j and sign $-$ for those that leaves the same node ($q_{ij}=0$ if there is no connection between nodes ij); C_j — the node discharge (consumption or supply from the reservoir), with the same convention of signs.

The functional relation between the head loss (h_{ij}) and the pipe line discharge (q_{ij}) is written, in general form, in the following way:

$$(2) \quad h_{ij} = H_i - H_j = R_{ij} q_{ij}^\alpha,$$

where: H_i and H_j represents the piezometric heads at nodes i and j ; R_{ij} — hydraulic resistance of pipe line ij . The exponent α varies between 1.8 and 2.

With the relation (2) the discharge becomes the expression :

$$(3) \quad q_{ij} = R_{ij}^{-1/\alpha} (H_i - H_j)^{1/\alpha}.$$

Introducing (3) in (1) there is obtained :

$$(4) \quad f_j = \sum_{\substack{i=1 \\ i \neq j}}^N R_{ij}^{-1/\alpha} (H_i - H_j)^{1/\alpha} + C_j = 0.$$

The relation (4), extended to all nodes with the unknown piezometric heads, leads to a system of nonlinear equations which, solved, determines the available piezometric heads at nodes and the discharges that flow in the pipe lines knowing the hydraulic resistance on pipe line and the consumption at nodes. Because the system (4) is nonlinear the solutions are obtained by Newton-Raphson method using the methodology shown in [1].

The available piezometric heads are compared with those that assure the service pressure. When the first ones have less values at numerous consumers it is raised the whole piezometric surface so that the service pressure be assured for all the consumers (nodes). If the available piezometric heads are less but in some isolated nodes there can be adopted as solution the raising of local pressure using a hydrophore installation then takes water from the exterior network by means of an air cushion tank.

3. The Studied Case

In Figs. 1 and 2 there are presented the distribution network of the town B in the present stage (Fig. 1) and under a concentrated form for the long term stage (Fig. 2).

The water supply is achieved from the surface source by using some reservoirs that assure a piezometric head $H_r = 210$ m WC (water column). The water supply network delivers a discharge of $0.457 \text{ m}^3/\text{s}$ (with fire $0.507 \text{ m}^3/\text{s}$), in present stage, with an increase in the long term stage to $0.623 \text{ m}^3/\text{s}$ (with fire $0.675 \text{ m}^3/\text{s}$).

These discharges are transported from the reservoirs up to node 1 of the water supply network by two influent conduits of $D_n = 500$ mm each and the length of 5 Km. The available piezometric head at the supply node 1, in the present making up and supply, results from extracting the head loss on the influent conduits (19.3 m WC) from the piezometric head of the reservoirs (210 m WC), having the values of 190.3 m WC.

The existing distribution network scheme comprises 114 bars, 75 nodes and 41 loops (Fig. 1) and the condensed long term scheme comprises 76 bars, 49 nodes and 28 loops (Fig. 2).

In Tables 1 and 2 there are presented the hydraulic resistance and necessary consumption discharges at nodes by solving Eq. (4) for the present and long term stages. In these tables the hydraulic resistances R_{ij} are given only for the cases when $i > j$, a sufficient demand for automatical programme of calculus.

Because in the water supply network nodes there must be assured service pressures having values between 16 and 20 m WC and the ground level in node 1 is of 174 m, there was simulated the change of the piezometric heads for available pressures in the supply node of 20 ; 25 and 30 m WC, values that practically can be taken into consideration as being possible to be obtained with existing influent conduits or can be achieved in future.

In Figs. 1 and 2 there were specified at every node, vertically, the piezometric head for service pressure assurance (the underlined value) and piezometric heads for the influent pressure $h_i = 20 ; 25$ and 30 m WC (respectively piezometric head of 194 ; 199 and 204 m WC), (only the inferior values of the service piezometric heads).

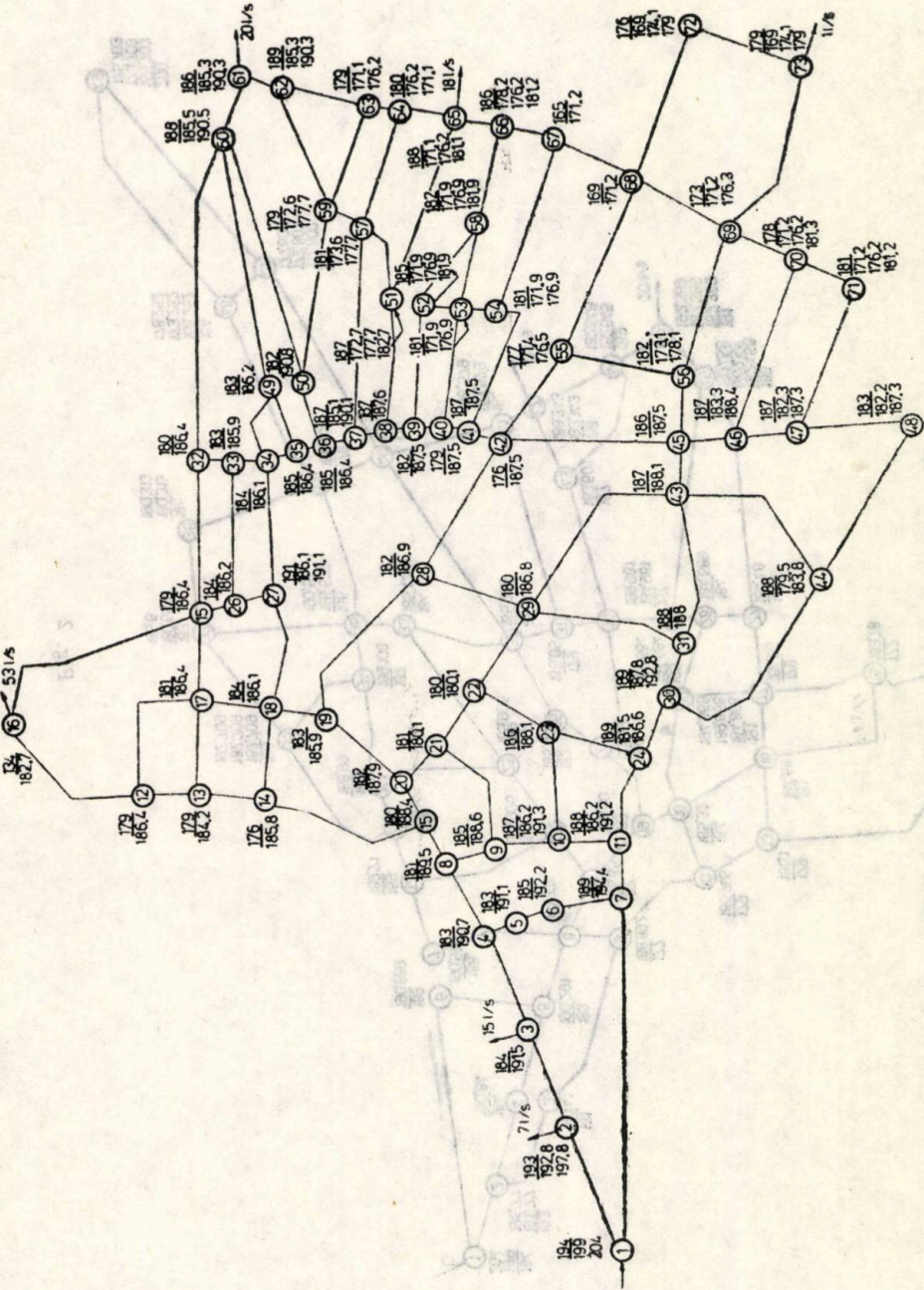


Fig. 1

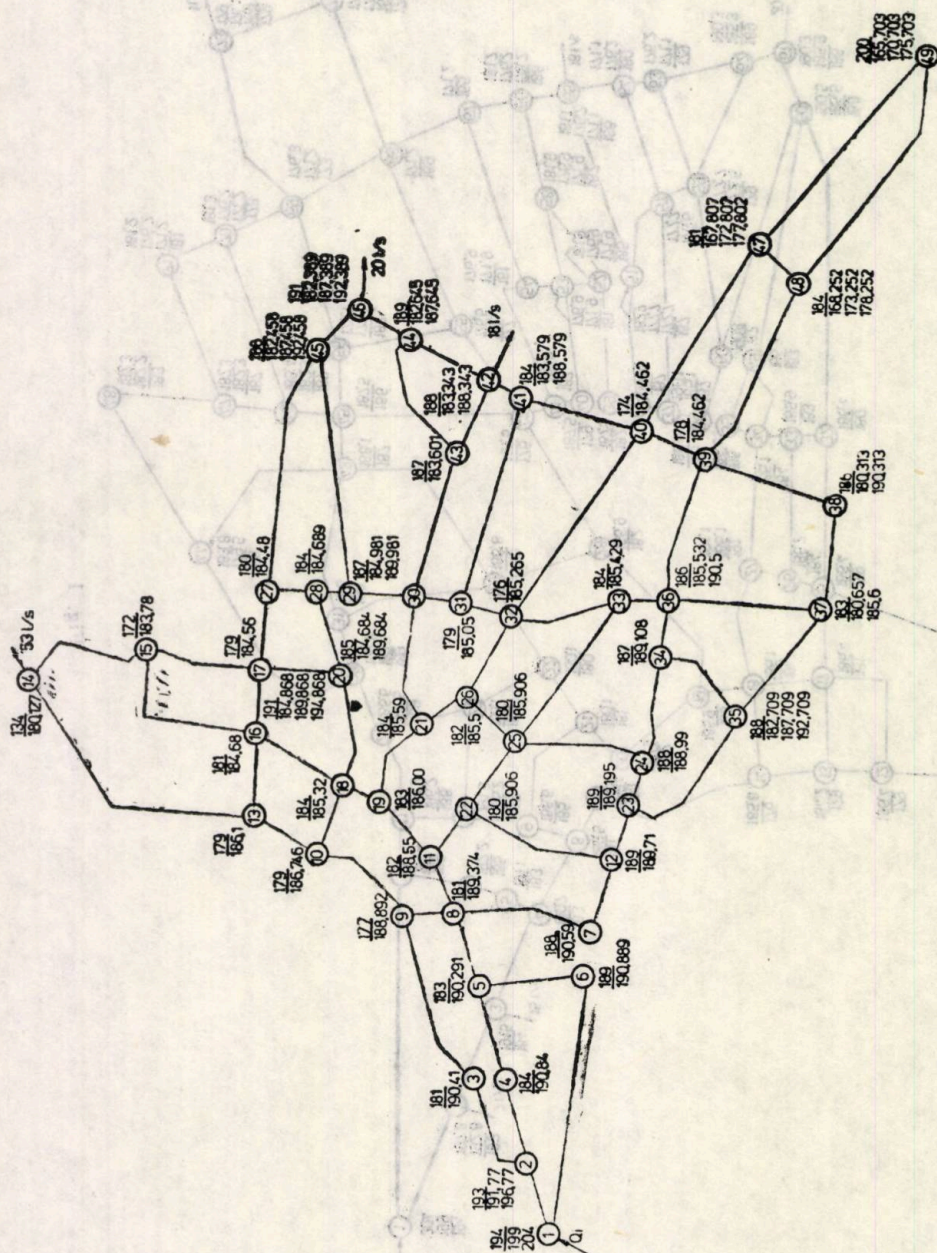


Fig. 2

Table 1

Node i	Node j (Hydraulic Resistance R_{ij})	Discharge C_j , [l/s]	Node i	Node j (Hydraulic Resistance R_{ij})	Discharge C_j , [l/s]
1	2 (22.1); 7 (35.5)	12.0	37	38 (941.7); 57 (452 894.0)	2.5
2	3 (27.2)	6.5	38	39 (100.4); 51 (580.1)	2.0
3	4 (18.4)	6.0	39	40 (75.3); 52 (92 910.5)	2.0
4	5 (60.3); 8 (21.8)	5.5	40	41 (100.4); 53 (5 734.2)	2.0
5	6 (129.3); 9 (1816.2)	4.5	41	42 (50.2); 54 (128 152.5)	2.0
6	7 (129.3); 10 (5 231.0)	4.5	42	45 (131.4); 55 (104 123.9)	6.5
7	11 (5.1)	12.0	43	44 (421 296.7); 45 (127.9)	8.0
8	9 (3 138.6); 15 (13.0)	4.5	44	48 (16 584.9)	4.0
9	10 (1 077.7); 21 (2 018.0)	7.5	45	46 (51 261.0); 56 (205.6)	4.0
10	11 (1 178.7); 23 (7 192.6)	7.0	46	47 (51 261.0); 70 (500 289.8)	2.5
11	24 (19.7)	5.5	47	48 (88 104.8); 71 (136 162.0)	2.5
12	13 (1 474.2); 16 (3 613.1); 17 (3 178.4)	13.5	48	60 (5 363.6)	3.0
13	14 (2 764.1); 17 (9 950.9)	6.5	49	59 (410 764.3); 60 (216 257.3)	3.5
14	15 (16 584.9); 18 (17 871.8)	8.5	50	57 (7 371.1); 65 (574.4)	2.0
15	20 (18.1)	6.0	51	53 (32 038.1); 58 (57 668.6)	2.0
16	25 (13 448.6)	13.5	52	54 (28 834.3); 58 (3 082.6)	2.5
17	18 (267.7); 25 (164.7)	10.5	53	67 (315 972.5)	2.5
18	19 (98.8); 27 (319.9)	9.0	54	56 (263 310.5); 68 (326 505.0)	2.5
19	20 (74.2); 28 (365.6)	7.5	55	69 (292.5)	2.0
20	21 (3 685.5)	5.0	56	59 (3 685.5); 64 (236 979.4)	2.5
21	22 (1 289.9)	4.5	57	66 (5 159.7)	1.5
22	23 (6 538.7); (1 934.9)	5.0	58	62 (13 820.8); 63 (184 317.3)	3.0
23	24 (5 884.8)	6.5	59	61 (333.5)	7.0
24	30 (15.3)	10.0	60	62 (463.2)	1.0
25	26 (593.0); 32 (247.1)	5.0	61	63 (1 222.9)	2.5
26	27 (948.9); 33 (315 972.5)	7.0	62	64 (277.9)	1.5
27	34 (652.6)	8.0	63	65 (648.5)	1.0
28	29 (741.1); 42 (297.0)	11.0	64	66 (370.5)	2.0
29	31 (9 808.1); 43 (19 349.1)	4.0	65	67 (426.1)	1.5
30	44 (18 427.7)	5.0	66	68 (1 009 556)	2.5
31	43 (120.5)	5.0	67	69 (741.1); 72 (658 276.1)	4.5
32	33 (840.8); 60 (2 927.5)	3.5	68	70 (48 057.1); 73 (21 191.9)	3.5
33	34 (504.5)	4.0	69	71 (48 057.1)	2.5
34	35 (672.6); 49 (1 112.4)	1.0	70	73 (7 923.9)	2.5
35	36 (269.0); 49 (4 791.2)	1.0	71		2.0
36	37 (773.5); 50 (48 057.1)	1.0	72		
			73		

Table 2

Node i	Node j (Hydraulic Resistance R_{ij})	Discharge C_j , [l/s]
1	2 (22.1); 6 (35.6)	15.5
2	3 (96.6); 4 (27.2)	13.5
3	4 (131 655.2); 9 (121.9)	14.5
4	5 (18.4)	9.0
5	6 (319.1); 8 (21.811)	10.5
6	7 (5.1)	17.5
7	8 (3 064.7); 12 (19.7)	10.0
8	9 (179 051.1); 11 (18.1)	12.0
9	10 (204.6)	13.0
10	13 (158.6); 18 (1 787.8)	9.5
11	19 (74.3); 22 (8 845.3)	7.5
12	22 (12 423.6); 23 (15.3)	11.5
13	14 (3 650.1); 16 (9 950.9)	17.5
14	15 (6 724.3)	16.0
15	16 (1 571.1); 17 (7 515.4)	18.0
16	17 (164.7); 18 (267.7)	16.5
17	20 (1 542.0); 27 (247.1)	14.5
18	19 (98.8); 20 (178.397)	11.5
19	21 (47.9)	7.5
20	28 (271.105)	11.0
21	26 (43.4); 30 (451.8)	9.5
22	25 (1 934.9)	10.5
23	24 (8.5); 35 (18 427.7)	5.5
24	25 (9 808.1); 34 (120.5)	7.5
25	26 (741.1); 33 (15 663.5)	14.5
26	32 (74.27)	8.0
27	28 (1 345.3); 45 (4 685.4)	15.0
28	29 (941.7)	7.0
29	30 (1 716.3); 45 (109 799.5)	14.0
30	31 (95.6); 43 (785.5)	13.5
31	32 (81.9); 41 (11 714.0)	10.0
32	33 (87.6); 40 (547.7)	15.0
33	36 (43.81)	14.5
34	35 (421 296.7); 36 (127.9)	4.5
35	37 (16 584)	5.0
36	37 (190 626.8); 39 (493.6)	8.5
37	38 (12 162.3)	4.5
38	39 (134 560.1)	2.5
39	40 (229.7); 48 (21 191.9)	8.0
40	41 (1 426.7); 47 (658 276.1)	14.0
41	42 (370.5)	12.0
42	43 (444.272); 44 (1 667.5)	9.5
43	44 (20 270.5)	13.5
44	46 (944.97)	14.5
45	46 (333.5)	19.5
46		3.0
47	48 (7 923.9); 49 (763 600.3)	7.5
48	49 (29 484.4)	8.0
49		5.5

4. The Analyse Results

After solving the equations (4) (73 equations) with the values of R_i and C_i from Table 1 (Fig. 1) there results the following observations:

a) For $h_i=20$ m WC there are not assured the service pressures in nodes 2, 10, 11, 24, 27, 30, 39, 44, 46, 47, 48, 51, 52, 53, 54, 55, 56, 57, 58, 59, 60, 62, 63, 64, 65, 66, 69, 70, 71, 72 and 73. In node 65 there is assured 3.1 m WC.

b) For $h_i=25$ m WC there are not assured the service pressures in nodes 24, 51, 52, 53, 54, 56, 57, 58, 63, 64, 65, 66, 70 and 71. In node 65 there is assured a service pressure of 8 m WC.

c) For $h_i=30$ m WC there are not assured the service pressures in nodes 51, 52, 54, 65, 66. In node 65 there is assured 13 mWC.

It results the necessity of raising the available pressure in influent node of the network in present stage at 30 m WC and local raising of pressure for the nodes 51, 52, 54, 65 and 66.

After solving the equations (4) (49 equations) with the values of R_i and C_i from Table 2 (Fig. 2 — the long term stage) there results the following observations:

a) For influent pressure $h_i=20$ m WC, the service pressure is not assured in nodes 2, 20, 29, 30, 35, 37, 38, 42, 44, 45, 46, 47, 48 and 49.

b) For $h_i=25$ m WC the service pressure is not assured in nodes 20, 38, 45, 46, 47, 48 and 49.

c) For $h_i=30$ m WC is not assured the service pressure in nodes 47, 48 and 49.

It results that for the planned extension of the network towards node 49 with pipe diameters $D_n=80$ mm and $D_n=100$ mm is not recommended beyond the nodes 39 and 40 (Fig. 2).

Table 3

Discharge m ³ /s	Pipe type mm	Head loss m WC	Piezometric head m WC	Available pressure m WC
Present stage (0.507)	2 $D_n=500$	19.3	190.7	16.2
	3 $D_n=500$	8.57	201.43	27.43
	2 $D_n=500 + 1 D_n=600$	5.87	204.14	30.14
	2 $D_n=500 + 1 D_n=800$	2.54	207.46	33.46
Long term stage (0.673)	2 $D_n=500$	34	176	2.0
	3 $D_n=500$	15.11	194.89	20.89
	2 $D_n=500 + 1 D_n=$ $=600$	10.33	199.67	25.67
	2 $D_n=500 + 1 D_n=$ $=800$	4.48	205.52	31.52

For raising the available pressure in influent node 1 it is cut down the head loss between reservoirs and the supply node supplementing the existing influent conduits (2 $D_n=500$ mm) with a new pipe line of $L=5$ Km. In order to establish the pipe diameter there were calculated the head losses of present discharge (0.507 m³/s) and the long term discharge (0.673 m³/s) for the existing influents conduits and for the existing influent conduits plus new pipe of $D_n=500$, 600 or 800 mm (Table 3).

For assuring a pressure $h_i=30$ m WC in node 1, there must be supplemented the influent conduits with a pipe having $D_n=600$ mm ($L=5$ Km) in the present stage and for the long term stage with the pipe with of $D_n=800$ mm. Taking into account that water supply in urban area is achieved for the long term stage it is necessary to supplement the influent conduits with a new pipe with $D_n=800$ mm.

5. Conclusions

1. The pressure state analysis in the existing water supply network in view of service pressure assurance can be efficiently achieved using the equations of discharge continuity at nodes expressed depending on the piezometric heads taken as unknowns (Eqs. (4)).

2. By simulating a corresponding number of schemes with the known piezometric heads of the influent node there can be determined the available piezometric heads in all the network nodes taking the necessary measures in order to assure the service pressure.

3. Although, in the studied example, the analysis was done for a network with a single influent node, it can be extended at network with many influent nodes considering as being known the pressures in these nodes. The calculus, using the node equations (4), will establish the pressure state in the rest of the network nodes.

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ANALIZA STĂRII DE PRESIUNE ÎN REȚELELE EXISTENTE DE ALIMENTARE CU APĂ ÎN VEDEREA ASIGURĂRII PRESIUNII DE SERVICIU

(Rezumat)

O problemă curent întâlnită la rețelele existente de distribuție a apei pentru centrele populate o constituie analiza stării de presiune în vederea asigurării presiunilor de serviciu la consumatori.

Această analiză se realizează eficient folosind ecuațiile de continuitate a debitelor la noduri (ecuațiile (4)) în care necunoscutele sînt cotele piezometrice. Cunoscînd la nodul (sau nodurile) de alimentare cota piezometrică (cotele piezometrice) se pot determina cotele piezometrice disponibile pentru celelalte noduri ale rețelei.

Folosind un număr suficient de simulări pentru presiunea în nodul (nodurile) de alimentare se pot stabili măsurile necesare pentru asigurarea presiunilor de serviciu în rețeaua de distribuție.

Cu un exemplu practic se ilustrează metodologia propusă.

UNE LOI DE DISTRIBUTION DES VITESSES DANS LES CANAUX TRAPÉZO-PARABOLIQUES

APPLICATIONS

PAR

ST. POPESCU, OVIDIU ALEXANDRESCU, DOBRICĂ LEU et ADRIAN POPIA

1. Introduction

Dans ce qui suit on généralise la loi de distribution des vitesses [2] pour les canaux trapézo-paraboliques [3], en indiquant aussi les techniques de mesure et l'interprétation des données permettant la déduction des constantes concernées.

Ensuite, en faisant appel à des méthodes numériques, on effectue les intégrales à l'aide des quelles on peut évaluer le débit (Q) et les coefficients de Coriollis (α) et de Boussinesq (β).

On déduit aussi une expression analytique, explicitée, pour les lignes isotaches.

Afin de résoudre opérativement les problèmes ci-dessus, on a utilisé un programme de calcul automatique en BASIC.

2. Loi de distribution des vitesses

En utilisant l'expression de la largeur du canal trapézo-parabolique à la cote z [3]:

$$(1) \quad B(z) = b + 2(az)^{1/n}$$

et suivant le raisonnement de [2] on déduit la relation :

$$(2) \quad u_{yz} = Ku_0 \cdot \frac{\ln \frac{z+k_f}{k_f}}{\ln \frac{H+k_f}{k_f}} \cdot \frac{\ln \frac{b/2+(az)^{1/n}+k_p-|y|}{k_p}}{\ln \frac{b/2+(az)^{1/n}+k_p}{k_p}} \cdot \frac{\ln \frac{H-\delta z}{k_a}}{\ln \frac{H}{k_a}}$$

Par l'intermédiaire des coefficients k_f , k_p et k_a la relation (2) prend en considération les forces de frottement du liquide avec le fond, les talus et respectivement l'aire atmosphérique.

Compte tenu que les rugosités k_f et k_p peuvent être connues avec une précision suffisante à l'aide de la littérature de spécialité, les seules constantes qui doivent être évaluées sont K , k_a et δ . Dans ce but on doit déterminer les triplets :

$$(3) \quad (u_{yz}^*, y, z)_i, \quad (k=1, 2, \dots, N; N \geq 2),$$

parmi les quelles se trouve obligatoirement le triplet qui correspond à la vitesse maximum dans la section u_0^* (par exemple, $(u_{yz}^*, y, z)_1 = (u_0^*, y_0, z_0)$).

Dans l'hypothèse de la distribution symétrique des vitesses, $y_0 = 0$; sur cette verticale la loi de variation de la vitesse devient :

$$(4) \quad u_{0z} = K u_0 \frac{\ln [(z+k_f)/k_f]}{\ln [(H+k_f)/k_f]} \cdot \frac{\ln [(H-\delta z)/k_a]}{\ln (H/k_a)}.$$

Si l'on impose la condition que z_0 soit capable de rendre maximum la vitesse u_{0z} , ($u_{0z}' = 0$), ainsi que le couple (u_0, z_0) satisfasse la relation (4), il en résulte le système :

$$(5) \quad (H - z_0 \delta) \ln \frac{H - z_0 \delta}{k_a} - \delta (z_0 + k_f) \ln \frac{z_0 + k_f}{k_f} = 0,$$

$$(6) \quad K \ln \frac{z_0 + k_f}{k_f} \ln \frac{H - z_0 \delta}{k_a} - \ln \frac{H + k_f}{k_f} \ln \frac{H}{k_a} = 0.$$

En utilisant les notations :

$$(7) \quad \begin{cases} \zeta(z) = \frac{z + k_f}{k_f}, \text{ avec } \zeta(z_j) = \zeta_j \text{ et } \ln \zeta(H) = x; \Lambda_0 = (z_0 + k_f) \ln \zeta_0, \\ \varphi_0(\delta) = \frac{\delta}{H - z_0 \delta} \cdot t(y, z) = \frac{0,5b + (az)^{1/n} + k_p - |y|}{k_p}, \\ \Psi_{ij}(\delta) = \Psi(z_i, z_j, \delta) = \ln \frac{H - \delta z_i}{H - \delta z_j} \text{ avec } \Psi_i(\delta) = \Psi(z_i, 0, \delta), \end{cases}$$

k_a et K résultent du système (5) et (6), seulement en fonction de δ , à savoir :

$$(8) \quad k_a = (H - z_0 \delta) \zeta_0^{-(z_0 + k_f) \varphi_0(\delta)},$$

$$(9) \quad K = \frac{x}{\ln \zeta_0} \left[1 + \frac{1}{\Lambda_0} \cdot \frac{\Psi_0(\delta)}{\varphi_0(\delta)} \right].$$

Afin d'évaluer la vitesse u_j dans le point de coordonnées (y_j, z_j) par une loi qui utilise le coefficient δ , on introduit (8) et (9) dans l'expression (1); avec les notations (7) on obtient :

$$(10) \quad u_j = \frac{u_0 \ln \zeta_j}{\Lambda_0 \ln \zeta_0} \cdot \frac{\ln t(y_j, z_j)}{\ln t(0, z_j)} \cdot \frac{\Lambda_0 \varphi_0(\delta) + \Psi_0(\delta)}{\Lambda_0 \varphi_0(\delta) - \Psi_0(\delta)} \cdot \frac{\Lambda_0 \varphi_0(\delta) + \Psi_{j0}(\delta)}{\varphi_0(\delta)}.$$

Le coefficient $\delta \in (0, 1)$, qui intervient dans les relations, (1) et (10), sera déterminé en fonction du nombre des mesures, (N) , comme suit : a) pour

$N=2$ on obtient, par voie numérique, la solution de l'équation (10) dans laquelle $j=2$ et u_j est substitué par la vitesse mesurée, u^* ; b) pour $N>2$, la valeur δ sera obtenue, conformément au principe de la méthode des plus petites carrés, de la condition de minimum de la somme :

$$(11) \quad \sigma = \sum_{j=2}^N (u_j - \bar{u}_j)^2,$$

où u_j est donné par (10).

3. Calcul du débit (Q) et des coefficients α et β

Les intégrales de surface pour l'évaluation de Q , du coefficient de Coriollis (α) et du coefficient de Boussinesq (β) [1] sont respectivement de la forme :

$$(12) \quad Q = \int_{(S)} u_{yz} dA, \quad \alpha = \frac{1}{A} \int_{(S)} \left(\frac{u_{yz}}{u} \right)^3 dA, \quad \beta = \frac{1}{A} \int_{(S)} \left(\frac{u_{yz}}{u} \right)^2 dA.$$

Si l'on effectue des calculs algébriques élémentaires on peut exprimer les relations (12) sous la forme équivalente :

$$(13) \quad Q = I_1, \quad \alpha = \frac{1}{A\bar{u}^3} I_3, \quad \beta = \frac{1}{A\bar{u}^2} I_2,$$

où

$$(14) \quad I_i = \int_{(S)} u_{yz}^i dA, \quad i \in \{1, 2, 3\}.$$

Grâce à la symétrie de la loi (1) par rapport à l'axe Oz , avec les notations :

$$(15) \quad p = \frac{Ku_0}{x \ln (H/k_a)}, \quad F_i(z) = \int_0^{\frac{b}{2} + (az)^{1/n}} [\ln t(y, z)]^i dy$$

on obtient :

$$(16) \quad I_i = 2p^i \int_0^H F_i(z) \left\{ \frac{\ln \zeta(z) \ln [(H - \delta z)/k_a]}{\ln t(0, z)} \right\}^i dz.$$

En mettant $F_i(z)$ sous la forme :

$$F_i(z) = k_p \int_1^{k_p t(0, z)} (\ln t)^i dt$$

et en intégrant, on obtient les expressions analytiques suivantes :

$$(17) \quad F_1(z) = k_p t(0, z) \ln t(0, z) - \frac{b}{2} - (az)^{1/n},$$

$$(18) \quad F_2(z) = k_p t(0, z) \ln^2 t(0, z) - 2F_1(z),$$

$$(19) \quad F_3(z) = k_p t(0, z) \ln^3 t(0, z) - 3F_2(z).$$

Ayant en vue leurs grande complexité, les intégrales (16), avec $F_i(z)$ données par les relations (17), (18) et (19), ont été effectuées par la méthode Simpson.

4. Détermination de l'expression analytique des isotaches

L'expression analytique explicite de l'isotache, précisée par la valeur u_k/u_0 , ($k=1, 2, \dots, N_{th}$), est déduite des relations (2) et (7), où l'on substitue u_y par u_k , en résultant :

$$t(y, z) = [t(0, z)]^{\Gamma_0(u_k/u_0)\Phi(z)} = t(0, z) - \frac{|y|}{k_p}$$

et finalement :

$$(20) \quad \frac{|y|}{k_p} = t(0, z) \left[1 - t(0, z)^{\Gamma_0(u_k/u_0)\Phi(z)-1} \right]$$

pour $z \in [z_{\min k}, z_{\max k}]$, où :

$$(21) \quad \begin{cases} 0 < z_{\min k} \leq z_0, & z_0 \leq z_{\max k} \leq H; \\ \Gamma_0 = \frac{z}{K} \ln \frac{H}{k_a} > 0; & \frac{1}{\Phi(z)} = \ln \zeta(z) \ln \frac{H - \delta z}{k_a}. \end{cases}$$

Si on considère, dans la relation (20), $y=0$, il résulte la suivante équation en z :

$$(22) \quad \Gamma_0 \frac{u_k}{u_0} \Phi(z) - 1 = 0, \quad \text{avec} \quad 0 < z \leq H.$$

Ayant en vue que $\lim_{z \rightarrow 0} \Phi(z) = \infty$ on obtient

$$(23) \quad \Gamma_0 \frac{u_k}{u_0} \Phi(z+0) > 1 \quad \text{pour} \quad \frac{u_k}{u_0} > 0.$$

Compte tenu de (4), (7) et (21) il résulte :

$$\Gamma_0 \frac{u_k}{u_0} \Phi(z_0) = \frac{u_k}{u_0};$$

donc

$$(24) \quad \Gamma_0 \frac{u_k}{u_0} \Phi(z_0) \leq 1 \quad \text{pour} \quad \frac{u_k}{u_0} \leq 1.$$

À l'aide des inégalités (23) et (24) on déduit que l'équation (22) admet, pour $0 < u_k/u_0 \leq 1$, une racine $z_{\min k}$; si, en outre,

$$(25) \quad \Gamma_0 \frac{u_k}{u_0} \Phi(H) \geq 1 \quad \text{pour} \quad \frac{u_k}{u_0} < 1,$$

l'équation (22) admet une deuxième racine, $z_{\max k}$; dans le cas contraire on adoptera $z_{\max k} = H$.

5. Exemple numérique

La loi de distribution des vitesses (1) et l'algorithme pour l'évaluation du débit (Q) et des coefficients (α , β) ont été vérifiés par voie expérimentale dans le cas d'un canal trapezoidal en béton qui a les suivants paramètres géométriques et hydrauliques: $n=1$, $b=0,6$ m, $a=m=0,89$, $k_p=0,001$ m, $k_r=0,902$ m, $H=0,45$ m, $Q=0,151$ m³ s⁻¹ (mesuré avec un déversoir triangulaire). Les vitesses u_{yz} ont été mesurées avec une micromoulinet hydrométrique dans les points de coordonnées (y , z), avec $z \in [0,05; 0,45]$ m, $|y| \in [0,00; 0,60]$ m, notés dans le Tableau 1; la vitesse maximum ($u_0=0,432$ m/s) a été mesurée au point 15 avec $y_0=0$ et $z_0=0,25$ m.

Utilisant une méthode numérique, basée sur programme BASIC, et à l'aide des données ci-dessous, on a obtenu les coefficients inconnus par la loi (1) à savoir: $\delta=0,861111$, $k=1,28396$, $k_0=0,0033$ m, $Q=0,149235$ m³ s⁻¹, $\bar{u}=0,33146$ m/s, $\alpha=1,14108$, $\beta=1,05223$; les vitesses théoriques, u_{yz} , ont été calculées à l'aide de la relation (1) (v. le Tableau 1).

Tableau 1

Données expérimentales et théoriques pour l'épure des vitesses

Le nombre d'ordre du point							
j	1	2	3	4	5	6	7
$ y $	0,0	0,10	0,20	0,30	0,40	0,50	0,60
z	0,45	0,45	0,45	0,45	0,45	0,45	0,45
u^*	0,340	0,316	0,321	0,281	0,326	0,296	0,268
u	0,341	0,333	0,323	0,312	0,297	0,276	0,240
j	8	9	10	11	12	13	14
$ y $	0,0	0,10	0,20	0,30	0,40	0,50	0,60
z	0,35	0,35	0,35	0,35	0,35	0,35	0,35
u^*	0,412	0,421	0,364	0,412	0,383	0,359	0,181
u	0,415	0,403	0,389	0,371	0,346	0,305	0,163
j	15	16	17	18	19	20	21
$ y $	0,10	0,20	0,30	0,40	0,50	0,00	0,10
z	0,25	0,25	0,25	0,25	0,25	0,15	0,15
u^*	0,432	0,403	0,383	0,374	0,229	0,422	0,388
u	0,417	0,399	0,373	0,332	0,218	0,414	0,396
j	22	23	24	25	26	27	28
$ y $	0,20	0,30	0,40	0,00	0,10	0,20	0,30
z	0,15	0,15	0,15	0,05	0,05	0,05	0,05
u^*	0,403	0,383	0,250	0,342	0,324	0,294	0,233
u	0,372	0,334	0,241	0,327	0,308	0,278	0,213

CALCULATION ELEMENTS CONCERNING ENERGETIC CONSUMPTION WITH SYSTEMS BY TRICKLE IRRIGATION IN MOLDAVIA

BY

M. LUCA and DAN PREPELIȚĂ

1. Introduction

An important criterion in watering equipment design at the trickle irrigation, that necessitates large quantities of material, is the energetic consumption exemplified under two forms: the energy included in the material that forms the watering equipment and the necessary energy for pumping water for pressure assurance of running. The watering equipment is formed of watering pipes, appliances and emitters. With the view to resisting to climate and mechanical factors action, the watering pipes with overground emplacement are made of plastics pipes, especially of low density polyethylen (PEld) and light type vinyl polychlorure (PVC-L). In trickle irrigation systems are pre-eminently used pipes with little diameters, made of PEld, which includes a large quantity of energy. In pumping systems the energetic consumption is also influenced by the value required by loss of pressure covering.

2. Characteristic Elements of Watering Equipment

The trickle irrigation systems designed and executed in N-E zone of Moldavia (Sirca system, Velnîța system, Maxut system) serve a series of fruit growing superintensive plantations organized in scheme 4×1 m (4 m distance between rows and 1 m between row trees [4]). The watering equipment is formed of watering pipes (lateral lines) of PEld with $D_i = 16 \dots 22$ mm (interior diameters) afferent to each row. The emitters of 4 and 6 l/h are disposed at 1 m intervals, at equal distance between fruit-trees. The watering pipes length is 80 ... 150 m, on an average of 100 m, resulting 2 500 m tube at each hectar of PEld.

The irrigated fruit growing plantation contains several kinds of apple trees, for which in pedological and climatological conditions of the Moldavian tableland have resulted from design, irrigation norms with 800 ... 1 000 m³/ha value (on an average 900 m³/ha) and brute norms of watering 270 ... 300 m³/ha (on an average 290 m³/ha). The watering norms have been applied in several stages the watering units of time being: 6; 8; 10 and 12 hours. The afferent surface of an emitter had $S = 1,4$ m².

3. The Hydraulic — Mathematic Model

The energy quantity included in plastics may be calculated with relation [1], [5]:

$$(1) \quad E_m = \frac{Lmc}{T},$$

where L is the watering pipe length need at each surface arranged unit, [m/ha]; m — tube mass at 1 m, [Kg/m]; c — total energy quantity included in 1 Kg of PEld, equal with 10 236 tcc/t or 83.3 KWh/kg [5]; T — using duration pipe. The tube mass depends on nominal diameter and its thickness. The emitters intervene with its our mass, distributed on each length unity of watering pipe, depending on its constructive structure. The pairs of values (E_{mi} , D_i) are remade mathematically to get correlation function, facilitating in this way the using of results. Energy consumption for water distribution by watering equipment is expressed by relation [3]:

$$(2) \quad E_p = \frac{\rho g Q H}{102 \eta} T,$$

or by introducing the irrigation norm, M and pressure in upstream extremity of the watering pipe, H_{up} :

$$(3) \quad E_p = k M H_{up}, \quad [\text{KWh/ha and year}],$$

where k is a constant which includes transforming factors (the invariable parameters from equation (2)) and has the value $k = 3.797 \times 10^{-3}$.

Considering the watering pipes with slope $I \cong 0$ (the row of the fruit-trees is situated on the level curve), the pressure in upstream section is given by relation:

$$(4) \quad H_{up} = H_{dw} + h_r,$$

where H_{dw} is the pressure of downstream section of watering pipe, necessary to last emitter running for a flow tolerance of $\delta_q = \pm 10\%$. The term h_r represents the energy loss along the watering pipe and it is calculated with relation [2]:

$$(5) \quad h_r = f J L,$$

where for a number of distributions $n > 50$, the correction coefficient is $f = 0.33$. The hydraulic slope J , for PEld tube, is given by relation [2]:

$$(6) \quad 100J = 0.0248 Q^{1.759} D_i^{-4.759},$$

in which Q and D_i are entrance flow, respectively the interior diameter of the watering pipe. For the flow watering pipe calculation must be known the emitters number n and their flow q according the relation:

$$(7) \quad Q = nq,$$

so that the flow imposed to an emitter results:

$$(8) \quad q = \frac{m_b S}{t_a},$$

where m_b is the brute norm of watering; S — the surface which correspond to an emitter; t_a — the application of the watering duration.

The microtube type emitters of PEld with interior diameter $D_i = 0.8$ mm, used in trickle irrigation systems in Moldavia tableland, have the hydraulic characteristics characterized by the equation:

$$(9) \quad q = 14.79 L^{-0.714} H^{0.696},$$

where L is microtube length; H — the service pressure and for the last emitter was considered $H = H_{av}$.

By calculation, it results a row of values pairs (E_{pt}, D_i) for a certain time of watering, t_a , which is remade mathematically under form of a correlation function.

4. The Obtained Results

The energy quantity included within plastics was determinated in function of the watering equipment characteristics of the irrigation systems by dripping in Moldavian tableland, namely: PEld tubes with 2 500 m/ha, exterior diameters of 16; 20; 25; 28; 31; 32; 35 mm and duration of using estimated to 20 years. For a variant of calculation it was taken into account also the energy included by microtube emitters, which were mounted meter by meter, presenting lengths of 0.42 ... 0.76 m, on an average equal to 0.60 m at which correspond a mass of 2.4 g/m and piece. The data remade according to Eq. (1) for a value of $c = 83.3$ KWh/Kg are presented in Table 1.

Table 1

The Energy Included in Watering Equipment Plastics Mass

No.	D_n mm	m Kg/m	m' Kg/m	E_m KWh/ha.year	E'_m KWh/ha.year
1	16	0.069	0.0714	718.46	743.45
2	20	0.086	0.0884	895.46	920.46
3	23	0.123	0.1254	1 280.73	1 305.73
4	25	0.134	0.1364	1 395.27	1 420.26
5	28	0.187	0.1894	1 947.13	1 972.13
6	31	0.209	0.2114	2 176.21	2 207.21
7	32	0.220	0.2224	2 290.75	2 315.74
8	35	0.236	0.2384	2 457.35	2 482.34

For the correlation function were choosed and verified three types of equations namely:

$$(10) \quad y = ab^x - \text{exponential type};$$

$$(11) \quad y = a + b \ln x - \text{logarithmic type};$$

$$(12) \quad y = ax^b - \text{power type},$$

at which the nearest correlation coefficient to unity was presented by the power type equation ($r = 0.989$). The obtained equations were:

$$(13) \quad E_m = 5.771 D^{1.719}$$

and taking into account emitters influence

$$(14) \quad E_m = 6.532 D^{1.686},$$

where D —the nominal diameter, [mm] and E_m , [KWh/ha year]; their graphical representation is given in Figs. 1 and 2.

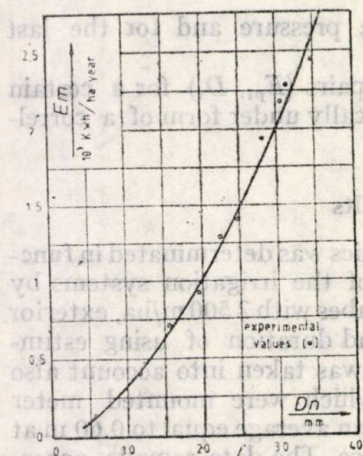


Fig. 1. — The energy variation included within watering pipe mass.

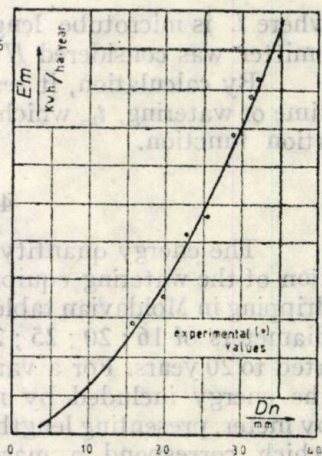


Fig. 2. — The energy variation included in watering pipe mass together with mounted emitters.

Analysing the values of Table 1 we can find that the emitters intervene with a increase of the energy included in plastics of about 1 ... 3% depending on diameter value.

To determine the required energy for water distribution there were used the following parameters: brute norm of watering, $m_b = 290 \text{ m}^3/\text{ha}$; the afferent surface of an emitter, $S = 1,40 \text{ m}^2$; time watering, $t_a = 4; 6; 8; 10$ and 12 hours; number of emitters on an watering pipe, $n = 100$ and pipe length, $l = 100 \text{ m}$.

Table 2

The Flow, Service Pressure at Emitters and Watering Pipe Flow

No.	t_a hours	q l/h	$H = H_{aw}$ m	n	Q l/s
1	4	10.15	25.61	100	0.282
2	6	6.76	14.28	100	0.187
3	8	5.07	9.46	100	0.141
4	10	4.06	6.86	100	0.113
5	12	3.38	5.27	100	0.094

From Eq. (8) has resulted the emitter flow for distribution of the norm m_b in the imposed time t_a and from Eq. (9) it was obtained the service pressure of the emitter, $H = H_{aw}$. The calculated data are presented in Table 2. Knowing the necessary pressure at the last emitter we have determined the pressure in upstream section of the watering pipe (Eq. (4)), where loss

energy was calculated with Eqs. (5) and (6). With these data, the necessary energy for water distribution on the watering equipment was calculated with Eq. (3), where $M=900 \text{ m}^3/\text{ha}$ (a calculation example being presented

Table 3

The Calculation Example of the Energy Necessary to Water Distribution on the Watering Pipes with $D_n=23 \text{ mm}$ ($D_i=18 \text{ mm}$)

No.	t_a hours	Q l/s	J m/m	h_r m	H_{aw} m	H_{up} m	E_p KWh/ha.year
1	4	0.282	9.36	3.12	25.61	28.73	98.17
2	6	0.187	4.54	1.51	14.28	15.79	53.96
3	8	0.141	2.76	0.92	9.46	10.38	35.47
4	10	0.113	1.87	0.62	6.86	7.48	25.56
5	12	0.094	1.35	0.45	5.27	5.72	19.54

Table 4

The Necessary Energy to Water Distribution on the Watering Pipes

No.	t_a hours	D_n/D_i , [mm]							
		16/12	20/16	23/18	25/22	28/23	31/26	32/27	35/30
1	4	161.0	106.2	98.2	92.6	90.8	89.4	89.1	87.8
2	6	84.5	57.9	54.0	51.3	50.4	49.7	49.6	49.2
3	8	54.0	37.8	35.5	33.8	33.3	32.9	32.8	32.6
4	10	38.1	27.2	25.6	24.5	24.1	23.8	23.7	23.6
5	12	28.6	20.7	19.5	18.8	18.5	18.3	18.2	18.1

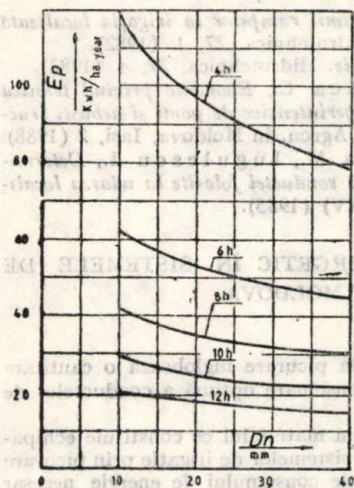


Fig. 3. — The necessary energy for water distribution on the watering equipment.

Table 5

The Equation (15) Parameters

No.	t_a hours	a	b	r
1	4	226.52	-0.285	0.953
2	6	110.19	-0.244	0.943
3	8	68.73	-0.226	0.943
4	10	47.55	-0.212	0.944
5	12	32.15	-0.200	0.944

in Table 3); the results depending on the watering time are centralized in Table 4. The values pairs (E_{pi} , D_i) for each watering time were remade for a correlation function (Eqs. (10) ... (12)), being obtained the following correlation coefficients corresponding to these equations; $r_{10}=0.913$, $r_{11}=0.937$ and $r_{12}=0.943$. In data processing it was adopted a correlation function power type of the form:

$$(15) \quad E_p = aD^b,$$

with the parameters a , b presented in Table 5 and with the graphical representation from Fig. 3.

5. Conclusions

The watering equipment for the trickle irrigation includes an important quantity of energy due to constituent material and to pressures that have to be assured for a good emitters running.

For watering equipment in a fixed scheme 4×1 m, the included energy in plastics (PEld) have values of 700 ... 2 500 KWh/ha a year, depending on watering pipe diameter.

The necessary energy for water distribution depends on watering time and the pipe diameter, by a correlation function power type. The energy value for PEld pipe diameters of 16 ... 35 mm is comprised within the interval 20 ... 100 KWh/ha a year.

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ELEMENTE DE CALCUL PRIVIND CONSUMUL ENERGETIC ÎN SISTEMELE DE IRIGAȚIE PRIN PICURARE DIN MOLDOVA

(Rezumat)

Echipamentul de udare al sistemelor de irigație prin picurare înglobează o cantitate importantă de energie, aspect ce devine un criteriu de dimensionare optimă a conductelor de udare.

Se prezintă ecuațiile de calcul ale energiei înglobate în materialul ce constituie echipamentul de udare (conducte de udare + picurătoare) specifice sistemelor de irigație prin picurare din Moldova. De asemenea se prezintă ecuațiile de calcul ale consumului de energie necesar asigurării presiunii de funcționare la picurătoare și a acoperirii pierderilor de sarcină pe echipamentul de udare. Ecuațiile de calcul, determinate prin analiza mai multor variante de diametre, debite, timpi de udare etc. au fost scrise, pentru o mai mare accesibilitate, sub formă matematică tip monom și reprezentate grafic.

LA CONSOMMATION D'EAU À LA CULTURE DU PÊCHER ET L'EFFET DE L'IRRIGATION DANS LA PLEINE BURNAS

PAR

CHIRIL POPESCU et C. NICOLESCU

1. Introduction

Le pêcher est une espèce qui demande relativement beaucoup de chaleur et dans les régions du nord de notre pays il peut souffrir à cause des températures basses. Sur les plaines et sur les plateaux où les hivers sont moins froids, les précipitations sont insuffisantes et il faut souvent appliquer l'irrigation. Au fur et à mesure que les surfaces irriguées augmentent, la demande d'eau augmente aussi. C'est pourquoi on utilise la méthode d'arrosage localisée (égouttement, tubes perforés, etc.) qui emploie l'eau plus économiquement. Aux arbres, la sécheresse n'est pas ressentie immédiatement, mais après une plus grande période de temps elle détermine la réduction de la récolte ou même la dessiccation des plantes [1]. Si l'on utilise des normes trop grandes d'arrosage on peut intensifier la croissance végétative arrivant à un murissement incomplet du bois et des domages pendant l'hiver [2].

Dans ce qui suit on présente l'étude de la consommation d'eau à une plantation classique et à une plantation intensive de pêcher et en même temps l'effet de l'irrigation par tubes perforés et par sillons sur la production des fruits.

2. Méthode de travail

Les recherches ont été organisées pendant la période 1975—1977 dans une plantation classique de pêcher ayant la distance entre les arbres de 5×6 m et aussi pendant la période 1980—1984 dans une plantation intensive ayant la distance entre les plantes 4×2 m. Ces plantations se trouvaient dans la pleine Burnas, localité Băneasa, département Giurgiu, sur un sol argileux avec 2,12% humus, pH=6,7, coefficient de flétrissement=10,9%, la capacité au champ=28,2% et la perméabilité modérée.

La moyenne des précipitations pendant la période des recherches a été de 515 mm, avec 39 mm ou 7% moins que la moyenne multiannuelle, et pendant la période de végétation ont tombé des précipitations totalisant 325 mm ou 63%. De la totalité des précipitations 33% ont été moindres ou égales à 10 mm, qui n'ont pas pu être utilisés par les arbres. On a enregistré des précipitations plus réduites que la moyenne dans les années 1976, 1983 et 1984. La température moyenne pendant les années de l'expérimentation a été de 10,3°C, avec 0,8°C moindre que la moyenne multiannuelle et pendant la période de végétation la température a été avec 1,2°C

plus réduite que la moyenne multiannuelle ($18,7^{\circ}\text{C}$). L'humidité relative de l'air a eu la valeur moyenne $68,6\%$ et l'évapotranspiration potentielle moyenne a été de 656 mm .

On a utilisé la variété du pêcher Jerseyland et l'irrigation a été appliquée sur des parcelles avec la surface de $0,5\text{ ha}$ dans les suivantes variantes : *A* — nonirrigué ; *B* — irrigué par des tubes perforés avec la distribution de l'eau par : b_1 — goulottes, b_2 — coupes à rayon de $0,5 \dots 1,0\text{ m}$; *C* — irrigué par sillons.

L'irrigation a été effectuée en utilisant l'eau du Danube, employant les normes indiquées dans le Tableau 2. La profondeur d'arrosage a été de 1 m et le plafond minimum $0,70\text{ IUA}$. Les coupes circulaires d'arrosage ont été réalisées manuellement ayant une profondeur de $15 \dots 20\text{ cm}$ mais les goulottes et les sillons d'arrosage ont été ouvertes d'une manière mécanisée à une distance de $0,75\text{ m}$ par rapport aux arbres, ayant la longueur de 75 m et la pente longitudinale $0,002$. Pour la distribution de l'eau aux sillons on a employé des conduits rigides d'aluminium munis d'orifices réglables à une distance de 1 m , ayant le débit initial $0,4\text{ l/s}$. À l'irrigation par des tubes perforés on a employé une installation de type fixe avec un débit moyen de l'ajoutage de 100 l/h et la pression de fonctionnement de $0,9$ baryes.

Par prélèvement des échantillons de sol on a déterminé la dynamique de l'humidité en établissant la date de l'application des arrosages. Pendant la végétation on a appliqué à la plantation étudiée les travaux d'entretien habituels.

3. Les résultats obtenus

En déterminant la consommation d'eau pour une période de trois années à une plantation classique de pêcher et pour cinq années à une plantation intensive (Tableau 1) on a constaté que sur un terrain nonirrigué pendant les six mois chauds de l'année (de 1 IV jusqu'à 1 X) la quantité d'eau consommée, en moyenne, a été de $3\,719 \dots 3\,760\text{ m}^3/\text{ha}$ avec des variations de $3\,411$ jusqu'à $3\,910\text{ m}^3/\text{ha}$. La densité des plantes n'a pas modifié d'une façon significative la consommation de l'eau sur le terrain nonirrigué. Pendant les mois avril et septembre on a enregistré la consommation la plus réduite d'eau ($431 \dots 450\text{ m}^3/\text{ha}$), et pendant le mois de juillet, la consommation la plus élevée ($930 \dots 944\text{ m}^3/\text{ha}$). Sur un terrain irrigué par tubes perforés la consommation d'eau augmente en atteignant $4\,421 \dots 4\,900\text{ m}^3/\text{ha}$ avec des variations de $4\,260$ à $4\,934\text{ m}^3/\text{ha}$. Elle est plus élevée pour la plantation classique avec $479\text{ m}^3/\text{ha}$ par rapport à celle intensive. La plus élevée consommation d'eau a été enregistrée pendant le mois de juillet ($1\,234 \dots 1\,240\text{ m}^3/\text{ha}$) et la plus réduite pendant le mois d'avril ($446 \dots 460\text{ m}^3/\text{ha}$). À l'irrigation par sillons la consommation d'eau a atteint, en moyenne, $6\,493\text{ m}^3/\text{ha}$, avec des variations qui vont de $6\,476$ à $6\,525\text{ m}^3/\text{ha}$. La plus réduite consommation d'eau a été enregistrée toujours pendant le mois d'avril à savoir, $510\text{ m}^3/\text{ha}$ et la plus élevée pendant le mois de juillet, $1\,610\text{ m}^3/\text{ha}$.

En étudiant la norme d'arrosage et d'irrigation (Tableau 2) on a constaté qu'à l'arrosage par tubes perforés on a utilisé, en moyenne, $960 \dots$

Tableau 1

La consommation de l'eau à une plantation de pêchers dans la plaine Burnas

Variante	L'année	La consommation mesuelle, [m³/ha]						Total m³/ha
		IV	V	VI	VII	VIII	IX	
Plantation classique								
Nonirrigué	1975	442	553	703	926	607	513	3 744
	1976	448	556	696	928	609	509	3 746
	1977	460	571	701	936	612	508	3 788
	Moyenne	450	560	700	930	610	510	3 760
Irrigué par des tuyaux perforés	1975	453	732	1 014	1 230	841	593	4 863
	1976	457	741	1 021	1 243	837	604	4 903
	1977	470	747	1 025	1 247	842	603	4 934
	Moyenne	460	740	1 020	1 240	840	600	4 900
Irrigué sur sillons	1975	508	914	1 403	1 604	1 392	655	6 476
	1976	507	922	1 391	1 600	1 398	661	6 479
	1977	515	924	1 406	1 606	1 410	664	6 525
	Moyenne	510	920	1 400	1 610	1 400	660	6 493
Plantation intensive								
Nonirrigué	1980	270	570	720	970	780	556	3 866
	1981	470	550	740	830	650	500	3 740
	1982	440	500	680	940	620	490	3 670
	1983	460	500	640	1 020	810	480	3 910
	1984	520	600	850	960	351	130	3 411
	Moyenne	432	544	726	944	642	431	3 719
Irrigué par des tuyaux perforés	1980	290	580	780	1 120	930	560	4 260
	1981	480	550	720	1 100	950	620	4 420
	1982	450	500	710	1 200	800	690	4 350
	1983	470	540	720	1 310	960	610	4 610
	1984	540	620	880	1 440	640	344	4 464
	Moyenne	446	558	762	1 234	856	565	4 421

Tableau 2

La quantité d'eau utilisée à l'arrosage et à l'irrigation pour une plantation de pêchers dans la pleine Burnas (moyenne 1975–1977 et 1980–1984)

La distribution d'eau	Variante	La norme d'arrosage m ³ /ha	Nombre d'arrosages	La norme d'irrigations m ³ /ha
Tuyaux perforés	Rigoles	170	8	1 360
	Creux de terrains	120	8	960
Sillons	Deux sillons sur chaque rang d'arbres	710	4	2 840

1 360 m³/ha tandis qu'à l'arrosage par sillons on a consommé 2 840 m³/ha. Dans le cas où la distribution de l'eau a été réalisée par coupes, aménagées autour du tronc de chaque arbre, la norme d'irrigation a été seulement de 960 m³/ha, c'est-à-dire avec 400 m³/ha moindre qu'à la distribution de l'eau par goulottes. La norme d'arrosage est pareille à celle employée dans une plantation de pomme de Maxut, département de Jassy [3].

Tableau 3

La manière dans laquelle se réalise la consommation d'eau à une plantation de pêchers de la pleine Burnas (moyenne 1975–1977 et 1980–1987)

Variante	La consommation totale d'eau m ³ /ha	L'eau du sol		Précipitations		Irrigations	
		m ³ /ha	du total %	m ³ /ha	du total %	m ³ /ha	du total %
Nonirrigué	3 739	1 326	35,5	2 413	64,5	—	—
Irrigué par des tuyaux perforés	4 660	1 104	23,7	2 413	51,8	1 143	24,5
Irrigué sur sillons	6 493	1 240	19,1	2 413	37,2	2 840	43,7

En étudiant la provenance de l'eau consommée à la plantation du pêcher de la pleine Burnas (Tableau 3) on a constaté que sur le terrain irrigué par des tubes perforés la consommation d'eau est avec 291 m³/ha plus grande par rapport à celle des terrains nonirrigués et à l'irrigation par sillons le dépassement est de 2 754 m³/ha. Les précipitations tombées dans cette zone assurent en moyenne 2 413 m³/ha ou 64,5% de la quantité

totale d'eau nécessaire sur terrain nonirrigué ; de la réserve d'eau du sol les plantes retirent 1 326 m³/ha ou 35,5% de l'eau nécessaire. À la culture irriguée par des tubes perforés l'eau qui provient des précipitations a assuré, en moyenne, seulement 51,8% de la quantité nécessaire totale; de la réserve existante dans le sol les plantes ont extrait 1 104 m³/ha ou 23,7% du total et par l'irrigation on a fourni, en moyenne, 1 143 m³/ha ou 24,5% du total. Dans le cas où on a appliqué l'irrigation sur sillons, les précipitations ont assuré seulement 37,2% du total de l'eau nécessaire du sol, la plante a utilisé 1 240 m³/ha ou 19,1% du total et par irrigation on a fourni

Tableau 4

La production de fruits à une plantation de pêcheurs dans la pleine Burnas (moyenne 1975–1977 et 1980–1987)

Variante	kg/ha	%	Différence	Signification	Utilisation de l'eau l/kg fruits
Nonirrigué	11 100	100,00	témoin	—	337
Irrigué par des tuyaux perforés	19 920	179,5	8 820	***	235
Irrigué sur sillons	19 200	172,9	8 100	**	338

DL–5%=2 150 kg/ha ; DL – 1%=3 949 kg/ha ; DL – 0,1%=8 481 kg/ha.

2 840 m³/ha ou 43,7% du total. Il résulte, donc, qu'à l'arrosage par tubes perforés la norme d'irrigation, dans cette zone, a été, en moyenne, de 2,4 fois moindre qu'à l'arrosage par sillons.

En déterminant la production des fruits enregistrée dans la plantation de pêcher (Tableau 4) on a constaté que sur le terrain irrigué on a obtenu une production qui dépasse avec 8 100 ... 8 820 kg/ha ou 72,9 ... 79,5% celle obtenue sur un terrain nonirrigué, prouvant l'importance de l'irrigation dans cette zone.

En calculant la consommation d'eau qui revient pour un kg de fruits on a constaté qu'à l'irrigation par tubes perforés on a obtenu la plus grande production de fruits (19 920 kg/ha) et la plus réduite consommation d'eau (235 l/kg de fruits avec 338 l/kg à l'irrigation par sillons).

4. Conclusions

1. Dans la pleine Burnas, où la moyenne annuelle des précipitations est de 554 mm et celle de la température de 11,1°C, la consommation d'eau à une plantation de pêcher a été, en moyenne, de 3 739 m³/ha sur terrain nonirrigué, 4 660 m³/ha lorsque l'irrigation a été effectuée par des tubes perforés et 6 493 m³/ha par irrigation par sillons. La plus grande consommation d'eau a été observée pendant le mois de juillet et la plus réduite pendant le mois de septembre.

2. La moyenne de la norme d'irrigation a été de 2 840 m³/ha à l'arrosage par sillons et 1 143 m³/ha lorsque la distribution de l'eau a été réalisée par des tubes perforés. Par l'aménagement des coupes pour retenir l'eau au long de chaque rangée on réduit la norme d'irrigation.

3. L'irrigation a augmenté la récolte de fruits avec 8 100 ... 8 820 kg/ha ou 72,9 ... 79,5% et la consommation de l'eau pour un kg fruits a été de 337... 338 l à l'irrigation par sillons ou sur terrain nonirrigué et de 235 l à l'irrigation par tubes perforés.

Requ le 29 février 1988

Institut Polytechnique, Jassy,

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foncières

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CONSUMUL DE APĂ LA CULTURA PIERSICULUI ȘI EFECTUL IRIGAȚIEI ÎN CÎMPIA BURNAS

(Rezumat)

S-au efectuat cercetări în localitatea Băneasa, județul Giurgiu, pe un sol molic luto-argilos, unde cad în medie 554 mm precipitații, iar temperatura medie anuală este de 11,1°C. Consumul de apă la două plantații de piersic, una clasică și alta intensivă, a fost, în medie, de 3 739 m³/ha pe teren neirrigat, 4 660 m³/ha cînd irigarea s-a realizat prin tuburi perforate și 6 493 m³/ha la irigarea pe brazde. Cel mai mare consum a fost constatat în luna iulie, iar cel mai redus, în septembrie. Norma de irigare a fost, în medie, de 2 840 m³/ha la udarea pe brazde și 1 143 m³/ha cînd distribuția apei s-a realizat prin tuburi perforate. Irigarea a sporit recolta de fructe cu 8 100 ... 8 820 kg/ha sau 72,9 ... 79,5%, iar consumul de apă pentru 1 kg fructe a fost de 338 l la irigarea pe brazde și 235 l la irigarea prin tuburi perforate.

4. Conclusions

1. Dans la plaine Burnas où la moyenne annuelle des précipitations est de 554 mm et celle de la température de 11,1°C, la consommation d'eau d'une plantation de pêcher a été en moyenne, de 3 739 m³/ha sur terrain nonirrigué, 4 660 m³/ha lorsque l'irrigation a été effectuée par des tubes perforés et de 6 493 m³/ha par irrigation par sillons. La plus grande consommation d'eau a été observée pendant le mois de juillet et la plus réduite pendant le mois de septembre.

ABOUT THE DEVELOPMENT OF THE SIZE OF BUILDING-ASSEMBLING ENTERPRISES

AURORA DIMA, M. DIMA and VICTOR ZAVATE

1. The dynamics and the complexity of economic and social development in present days has put in evidence a new trend in what concerns the size of building-assembling enterprises.

In order to put in evidence the tendency of the size development of building-assembling enterprises in what follows we have in view elements concerning both enterprises, or companies, belonging to the capitalist system and also to the socialist one, from a large number of european countries. A part of these elements are analysed in the present paper, processed in a manner which point out the development tendency of enterprise size in the last decade and in the first half of the present one.

2. Comparable data from this period of time concerning the development of the number of enterprises related on size types have been taken into account: very small, small, middle size, large and very large size. These have been established depending on the number of the employees. The enterprise types have been stated conventionally for the sake of comparing and interpreting needs.

It has been followed, at the same time, the evolution of the achieved output and the evolution of the employees number on enterprise types.

Among the countries for which the elements refering the development of the size of building-assembling enterprises have been studied, comprising the period 1970—1984, having in mind the lack of space, the item presents only four countries, namely: German Federal Republic, Great Britain, the Hungarian People's Republic and the Polish People's Republic.

3. As regards the German Federal Republic, the dynamics of the three indicators above mentioned, in the considered period, for different types of enterprises, are evidenced through the elements given in the Figs. 1...3.

As regards the number of the building enterprises for different types of size, in the analysed period, it has been observed that the greatest weight (about 90%) is hold by the very small enterprises (having no more than 50 employees) and the small ones (having between 50 and 100 employees) with a slight increasing tendency of their number.

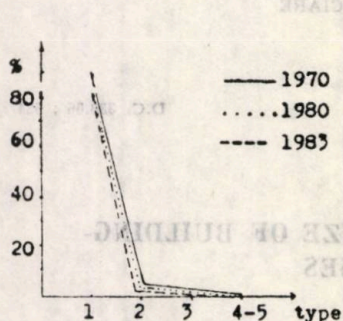


Fig. 1

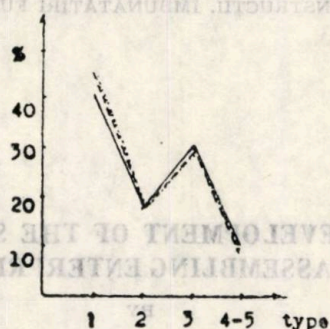


Fig. 2

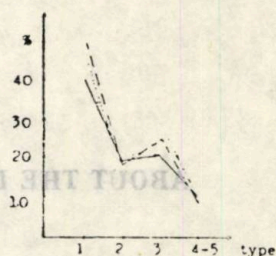


Fig. 3

The middle size enterprises (having between 100 and 500 employees) tend to stay constant as regards their number, occupying about 3%, and the large and very large enterprises (having between 500 and 1 000, and respectively over 1 000 employees), interfere with a very small weight, practically under 1%, with a decreasing tendency as it results from Fig. 1.

As regards the value of the building output, over 60% is achieved in very small and small enterprises with increasing tendency; about 30% is carried out in middle size enterprises with a stabilisation trend and only about 10% from the value of the building output is carried out in large and very large size enterprises (Fig. 2).

The labour power busy in the building-assembling field, in the very small and small size enterprises, holds a weight of over 60% with a growing tendency after 1980. The labour power busy in the middle size enterprises holds a weight of over 20%, with a stabilisation tendency, and the busy one from the large and very large enterprises holds a weight up to 10% with a slight decreasing but permanent tendency (Fig. 3).

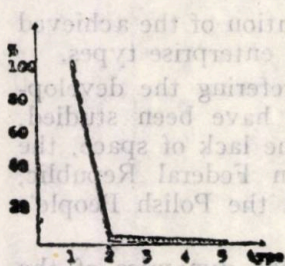


Fig. 4

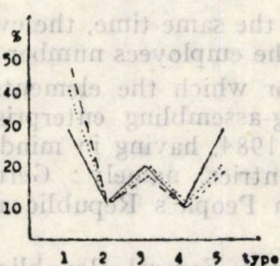


Fig. 5

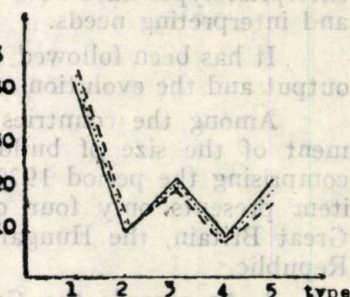


Fig. 6

4. The evolution of building enterprises from Great Britain, on the basis of the dynamics study of the some indicators, is evidenced in the Figs. 4... 6.

From their outlook it results that regarding the number of enterprises, about 96% are very small and small with a fast increasing tendency of

these from the category with up to 5 employees and with a decreasing tendency of those having 6 ... 49 employees; the weight of the average enterprises decreases from 1.60% in 1970 to 0.80% in 1980, with a slight decreasing trend ahead. The weight of the number of large and very large enterprises is keeping under 1% with a slight but permanent decreasing tendency (Fig. 4).

The output achieved in small and very small enterprises represents a weight of over 50% with a moderate increasing tendency that obtained in average enterprises represents about 20% weight with a stabilisation tendency; the output performed in large firms holds a weight of about 9% with a decreasing tendency and the performed output in very large enterprises represented in 1970 about 30%, whereas in 1983 reaches until 17% (Fig. 5).

The labour power busy in constructional works in Great Britain follows approximately the same development process like for output resulting from the graphical representation in Fig. 6.

5. In the Hungarian People's Republic (H.P.R.), in this period, numerically, the largest weight belongs to the middle size enterprises, namely about 50%; on the next place are situated the very large enterprises, having a 20% weight, followed by the large ones with about 14% weight; on the last place are situated the very small and small enterprises which in 1970 held 32% and in 1982 less than 10% with a decreasing tendency afterwards (Fig. 7).

The volume of the achieved output in very small and small enterprises represents, at the 1970 year level, about 3%, reaching in 1982 to be still 0.38%, thus decreasing more than seven times; the achieved level in middle size enterprises occupies a weight of about 11 ... 12% with stabilization tendency or slight increase; in the large enterprises is carried out about 12% from the output with a slight increasing tendency and in very

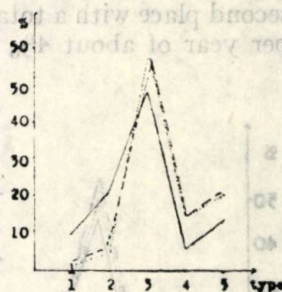


Fig. 7

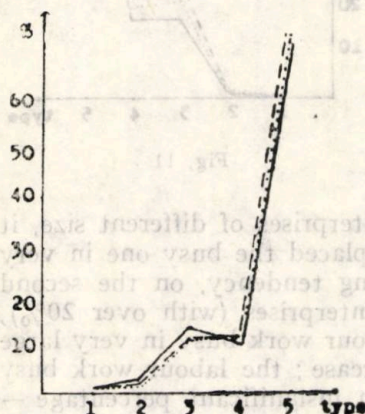


Fig. 8

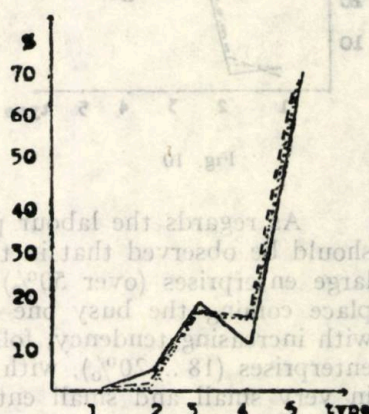


Fig. 9

large enterprises is performed 75% output with the stabilization or even a slight decreasing tendency (Fig. 8).

The dynamics of the labour key personnel over the some period shows that in H.P.R. the biggest part of the labour power is busy in the very large enterprises — over 65%, with a stabilization tendency; it is followed by the large enterprises which hold about 15% from the labour power, with the tendency of slight decrease; the middle size enterprises hold about 17% from the labour power with a stabilization or slight increasing tendency; in the small size enterprises the labour work busy decreases from 4% in 1970 to 0.60% in 1980 and shows afterwards a slight increasing tendency, so that in 1982 reaches until 0.70%; the very small enterprises are not to be found in the statistical data recorded after 1970 (Fig. 9).

6. In the Polish People's Republic (P.P.R.), the very small enterprises held in 1980 a weight of about 4% related to their total number, reaching in 1981 to 5%, afterwards decreasing in 1982 to 3%, that is a nonconclusive evolution.

The small size enterprises occupy a weight of about 5%, with a tendency of slight increase whereas the middle ones are performing a slight but permanent increase, holding the most important weight — about 50%; the large size enterprises occupy about 17% from the whole, presenting slight decreasing tendency, whereas the very large enterprises occupy the second place with a total weight of over 20%, but having a decrease tendency per year of about 4% (Fig. 10).

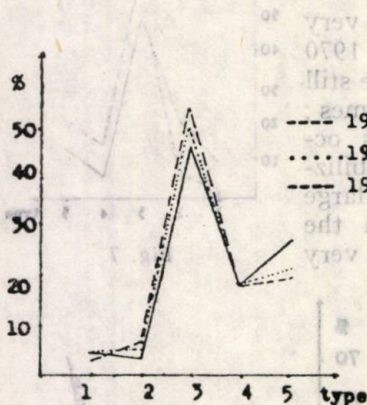


Fig. 10

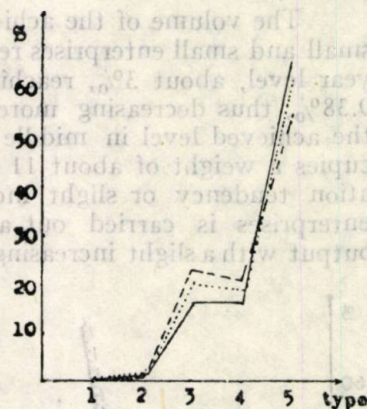


Fig. 11

As regards the labour power busy in enterprises of different size, it should be observed that in the first place is placed the busy one in very large enterprises (over 50%) with a decreasing tendency, on the second place coming the busy one in middle size enterprises (with over 20%), with increasing tendency, followed by the labour work busy in very large enterprises (18 ... 20%), with a moderate decrease; the labour work busy in very small and small enterprises holds an insignificant percentage — under 1%, with decreasing tendency (Fig. 11).

As regards the value of the achieved output on types of enterprises in the P.P.R., they were not available statistical data. Generally, we can suppose it follows the same evolution like the labour work busy in different types of enterprises (Fig. 11).

Conclusions

The evolution tendency of the size of building-assembling enterprises for the analysed period of time and potentially in the future, is the following :

1. In the capitalist countries, the very small and small building enterprises hold the most part of the weight — about 90% from the whole, in number, and also in the value of the achieved output and busy labour work, with an increasing tendency in the future. The large and very large enterprises occupy insignificant weights, about 1%, with an ever decreasing tendency (G.F.R., England, Austria, Netherlands, Belgium, France, Sweden).

2. The first place in the socialist countries — in number — is occupied by the middle size enterprises (with 100 ... 499 employees) having a weight of over 50% from the whole, with a moderate increase or stabilization tendency.

As regards the value of the achieved output and the busy staff for different types of enterprises, the maximum weight is held by the very large enterprises, namely 65 ... 70% and even more ; the middle size enterprises hold a weight up to 20%, whereas the very small ones hold an insignificant weight with a permanent decreasing tendency. The small size enterprises (50 ... 100 employees), at the same time, hold very low weights but their number tends to increase slightly after 1980.

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CONSIDERAȚII PRIVIND ORIENTAREA DEZVOLTĂRII ÎNTEPRINDERILOR DE CONSTRUCȚII-MONTAJ

(Rezumat)

Studiul întreprins pune în evidență, cu ajutorul unor parametri reprezentativi, orientarea dezvoltării întreprinderilor de construcții-montaj pe plan mondial, furnizând astfel prețioase elemente de prognoză pentru organizarea activității din acest sector.

As regards the value of the achieved output on types of enterprises in the P.R.R. they were not available statistical data. Generally, we can suppose it follows the same evolution like the labour work busy in different types of enterprises (Fig. 11).

Conclusions

The evolution tendency of the size of building-assembling enterprises for the analysed period of time and potentially in the future is the following:

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2. The first place in the socialist countries — in number — is occupied by the middle size enterprises (with 100...499 employees) having a weight of over 50% from the whole, with a moderate increase or stable action tendency.

As regards the value of the achieved output and the busy staff for different types of enterprises, the maximum weight is held by the very large enterprises, namely 65...70%, and even more; the middle size enterprises hold a weight up to 20%, whereas the very small ones hold an insignificant weight with a permanent decreasing tendency. The small size enterprises (50...100 employees), at the same time, hold very low weights but their number tends to increase slightly after 1980.

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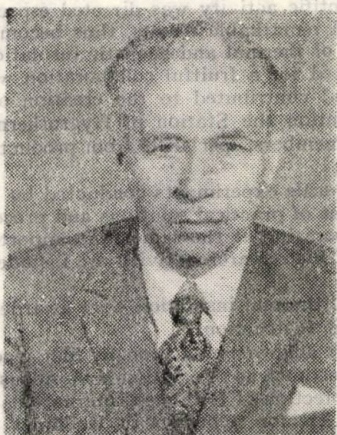
1. "Annuaire statistique de France, 1981 (INSEE) 42 (1982)
2. "Polish Statistical Yearbook of the Council of the European Communities, 1981 (INSEE) 42 (1982)

CONSIDERATION PRINCIPALE DES ENTREPRISES DE CONSTRUCTION

(Résumé)

Évolution tendancielle de la taille des entreprises de construction pour la période analysée et potentiellement à l'avenir. Les entreprises de très petite et petite taille occupent la plus grande part du poids — environ 90% du total en nombre, et aussi en valeur de la production réalisée et en effectif salarié, avec une tendance à l'augmentation à l'avenir. Les entreprises de grande et très grande taille occupent des poids insignifiants, environ 1%, avec une tendance à la diminution constante (États-Unis, Angleterre, Autriche, Pays-Bas, Belgique, France, Suède).

CONSTRUCȚII. ÎMBUNĂTĂȚIRI FUNCIONARE



VIRGIL FOCȘA

1923—1987

It is profound grief that we report the sad demise on October 5, 1987, of the eminent scientist and distinguished researcher in the domain of civil and industrial constructions, Prof. Dr. Eng. Virgil C. Focșa.

Born on July 26, 1923, in Huși, he attended primary and secondary school in this town. After receiving „le diplôme de baccalauréat” in 1942 he attended the courses of the Civil Engineering Faculty within the Polytechnical Institute of Jassy, graduating in 1946 with *Mağaa cum Laude*.

In the beginning he worked as a civil engineer for the Jassy Railway Department until 1947, when he joined the Civil Engineering staff as assistant. In 1951 he was promoted associate professor and afterwards, in 1969, professor at the Department of Building Engineering, position which he held until recently.

Prof. Focșa's lectures were always attended with permanent interest by the students and graduates in Civil Engineering due to their highly scientific standard.

Throughout the forty years dedicated with intense self abnegation to the higher education in Civil Engineering at the Polytechnical Institute of Jassy, Prof. V. Focșa held various important positions: head of department (1963—1976), (1984—1987), deputy dean (1963—1969), dean (1976—1984), performing an ample and praiseworthy organizing activity for the development of the Civil Engineering Faculty in Jassy, for promoting and continuously improving the integration of education with research — design and production activities, for a permanent improvement of the whole instructive — educational process.

In 1967 Prof. V. Focșa received the Ph. D. degree and, since 1969, he was nominated scientific advisor for Building Engineering, supervising the activity of 37 doctor's trainers.

Prof. Focșa's educational and scientific activity is reflected in 251 papers published by many prestigious scientific journals both in Romania and abroad.

In 1975 he published *Hidrotermica și Acustica Clădirilor* (Hygrothermics and Building Acoustics) synthesizing for the first time in Romanian original researchs and studies concerning building environment and energy conservation in constructions. The book enjoyed widespread appreciation both in our country and abroad (England, D.R.G., Holland, Poland).

In 1976 he was co-author of *Tratat de Construcții Civile* (Treatise on Civil Constructions), the first in Romanian, and in 1985 he published *Clădiri în climat cald* (Buildings in Hot Climate) containing numerous original elements.

The large number of scientific works written in collaboration bear witness to the author's close connection with specialists from education, research and production institutes and to the permanent care of stimulating younger collaborators in teaching and scientific activity.

Prof. V. Focșa's scientific activity was directed especially towards building physics, domain in which he obtained remarkable results, thus becoming an expert in Romania and abroad, especially in the field of thermal and acoustic insulations.

In this respect he worked for a fruitful collaboration with the Romanian Institute of Research in Constructions, and contributed to the creation of a rich experimental ground-work (if we were only to mention the Station of Hygrothermic Research in Constructions, by which many teaching staff members, specialists from production and research units, doctor's trainers have benefited).

The laboratory and full scale experiments conducted by Prof. V. Focșa enabled their correlation and the development of practical solutions and procedures which have been effectively implemented, while elaborating at the same time important national building codes.

The original elements of his research activity are included in 9 patents and innovator's certificates.

His works in the field of thermal insulation were awarded prizes, in 1964 and 1967, by the Ministry of Education and Instruction.

Prof. Focșa's scientific achievements brought him recognition certified by his membership in numerous organizations such as : Commission on Acoustics of the Romanian Academy, Commission on Physics and Protection of Buildings International Building Council (CIB W-40), Technico-Scientific Council CAER for Acoustics in Buildings a.s.o.

A party member from 1962, Prof. V. Focșa performed with self denial and responsibility, many leading duties within the party organs of the Civil Engineering Faculty, Polytechnical Institute and University Centre of Jassy.

He served as a deputy of the city and county people's councils, president of County Committee of Engineers, member of the Municipal Urban Planning Commission, member of Jassy County Financial Commission, vicepresident of Jassy Scientists' Association, „Structural Journal“ editorial staff member, etc.

As a recognition of his professional, scientific socio-political activity, he was awarded numerous Romanian orders and medals.

His exemplary self-abnegation concerning the scientific and teaching activity was manifest to the last period of his life when, although very ill, he worked assiduously in elaborating two treatises: *Urban and Industrial Acoustics* and *Architectural Acoustics*, which are being published by the Technical Publishing House and SRR Academy Press, respectively.

Prof. Focșa's self-devotion to Romanian higher learning ideals flourishing, his exceptional qualities as a professor and scientific researcher with remarkable results, contributing to over forty graduate series thorough training, his political-social activity, his humaness ensure his unforgettable memory remain with all those who knew and appreciated him as a renowned personality among the gallery of great scholars of Romanian civil engineering education.

Prorector, Assoc. Prof. Dr. Eng. C. Șerbescu

RECENZII

BOOK REVIEWS

РЕЦЕНЗИИ

PULLAR-STRECKER P., *Corrosion Damaged Concrete Assessment and Repair*. Butterworths Scientific Ltd., London, 1987, 99 pp.

The book by Peter Pullar-Strecker, edited by the Butterworth Publishing Company, deals about the corrosion damaged concrete, the assesement and repair of these damages.

Some of the reinforced concrete structures are affected in their durability, which needs for early repair.

The book is a real help in solving such problems and gives many indications how to do this in the commonest of concrete durability problems. So it is pointed out how to understand the phenomena which lead to the damages and how they can be put right.

The carefully chosen biliography helps to extend the knowledge deeper of the subjects outlined.

It is the merit of the author and the Butterworth Publishing Company to extend the knowledge in this field.

T. Silion

MEIGH A. C., *Cone Penetration Testing Methods and Interpretation*. Butterworths Scientific Ltd., London, 1987, 141 pp.

The knowledge of the real behaviour of soils in their natural site is of great importance in the design of foundations. In this direction the development of *in situ* procedures is desired from all engineers acting in the field of soil mechanics.

The book presented by A. C. Meigh and edited by Butterworth Publishing Company is of great interest and represents a valued contribution in the problem of cone penetration testing. Are analysed the equipment, the procedure and the use of CPT results. Of great practical utility is the determination of the parameters in cohesionless and cohesive soils, the correlation between different penetration tests and also in other materials. In the same time the book deals with the use of these tests in the design of foundations, considering safe bearing capacity and allowable bearing capacity related to tolerable settlements for different solution like footings, rafts, piles and other applications. The use of piezocones permits the measurement of the porewaterpressure.

The great number of references elevates the value of this book. It is the merit of the author and the Butterworth Publishing Company that this book appeared for the need of engineers.

T. Silion

KLENGEL K. J., WAGENBRETH O., *Ingenieur Geologie*. VEB Verlag für Bauwesen, Berlin, 1987, 244 S.

Die zweite Auflage des Buches *Ingenieurgeologie für Bauingenieure* von Prof. Dr. sc. nat. K. J. Klengel und Doz. Dr. rer. nat. habil. O. Wagenbreth zeugt von dem Wert dieses Buches und dem Interesse mit welchem es von der Fachwelt aufgenommen wurde.

Der Inhalt ist besonders ausgewählt und auf die Probleme der Bauingenieure abgestimmt.

Die Erweiterungen die diese zweite Auflage erfahren hat erschöpfen die meisten Probleme die für die Praxis von Bedeutung sind.

Es ist das Verdienst der Verfasser der Fachliteratur damit ein wertvolles Lehrbuch für die Studenten und Bauingenieure geschaffen zu haben.

Es wird besonders unterstrichen daß der Verlag mit diesem Buch eine Ausgabe erzielt hat die wesentlichen Anforderungen gerecht wird.

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T. Silion

KLEMDER, K. J., WAGENBRETT, O., Ingenieurbau Geologie, VEB Verlag für Bauwesen, Berlin, 1987, 214 S.

Das Buch, Auflage des Buches Ingenieurbau Geologie für Bauingenieure von K. J. Klemder und O. Wagenbrett, ist ein wertvolles Buch, das den Ingenieuren mit Interesse an Geologie ein wertvolles Buch sein wird.



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